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Save the Golden Band of Ocean Beaches!

New Jersey beaches for 125 miles have a taxable valuation of $4,000,000 a mile—Every year the ocean is trimming down this golden band while a public is unaware of the loss, and administrations fail to undertake the simple preventive work necessary

By Victor Gelineau

Director, State Board of Commerce and Navigation, Jersey City, N. J.

BEACH PROPERTY of immense value was destroyed this past winter by a series of onshore storms which swept the Atlantic seaboard from Florida to the outermost coasts of Newfoundland and Labrador. Estimates of the values confiscated by the sea are unreliable because no agencies exist for appraising them. Any count of the beach values that were hazarded is nearly as uncertain because these are confined to structures built for amusement and recreation, and people have selected and abandoned their recreation spots repeatedly from decade to decade, creating new and destroying old values.

The seaboard of New Jersey has been extensively developed, as it has generally fine beaches for ocean bathing and many protected inlets for boat harbors, affords good opportunity along the coast for fishing and is near large centers of population in metropolitan New York and Philadelphia. This development varies from the costly concentrations of recreation structures, as at Asbury Park and Atlantic City, to little settlements and scattered strips of beach bungalows all along the mainland and barrier islands. The map (Fig. 2) and the views (Figs. 1 and 4) indicate numbers and types of these beach developments. They extend 125 miles from Atlantic Highlands at New York harbor to Cape May at the mouth of Delaware Bay. The tax ratables of this strip of beach properties in 1932 exceeded $550,000,000 or $4,000,000 a mile of beach. These are the values hazarded by storms that destroy beaches, for it is the beach patronage that creates all investments. When the beaches go most values in the coast towns go with them.

Fig. 1—The barrier beach north from Monmouth Beach, showing inroads of sea and futile private efforts to protect property. Thirty years ago costly summer residences bordered the beach. Last winter's waves surged across the land spit into the Shrewsbury River.

Beach damages are constant and recurring. The storms of last fall and winter, because they were exceptional gales and followed so closely in succession as to have a cumulative destructive effect, assume the importance of disaster. Their effect, as shown by the wreckage appearing in Figs. 3, 5 and 6, are merely magnifications of the effects of every onshore storm. Beach damages are not the result of exceptional

off-sea hurricanes, but come with every recurring storm of fall, winter and spring. The beaches are under constant attack, both local and general.

Conditions in New Jersey may be cited as typical. Such surveys as are available indicate that since 1840 the shore line of the New Jersey ocean front has receded at an average rate estimated at 1½ ft. per year. This is not uniform and is powerfully affected by the notably great changes that have taken place on some of the low-lying points adjoining some of the inlets (Fig. 7). In a few sections the land area has tended to gain, but unquestionably with few exceptions the land areas

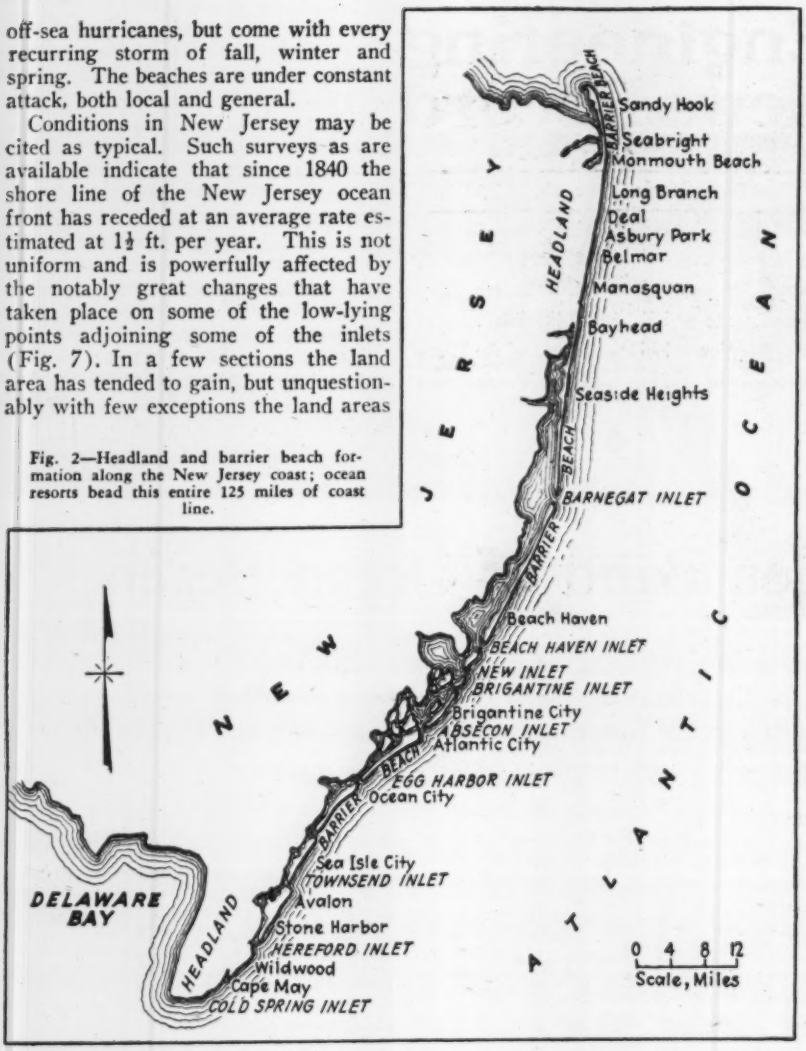


Fig. 2—Headland and barrier beach formation along the New Jersey coast; ocean resorts bead this entire 125 miles of coast line.

tend to diminish under the influence of the sea forces. This applies just as truly to the entire belt of sandy beaches from Montauk to southern Florida as to the New Jersey frontage. Subsidence or emergence may be in progress, but the rate of change is so slight as to escape detection upon comparison of present-day levels with those taken 25 or 30 years ago.

These facts have particular significance, first, in the respect that they are widely unrealized as facts, and second, that when realized they come to be appraised as irresistible phenomena. Realization is difficult to create because the great beach-using public ordinarily sees the ocean only during its most idle moments. Only the chance visitor sees the happenings pictured in Figs. 3, 5 and 6, as these storms do not occur during the season when the beaches are patronized by visitors. It is only the lighted and dressed show-window that the customer sees. Herein is the explanation in part why people's concern in beach protection as a public work is as rare as is explained later.

In respect to the second factor mentioned there is no occasion for an attitude of discouragement. It is entirely a question of economics and balancing of benefits. Can a given community afford to provide sea defenses, or can it afford to postpone remedial measures, facing as the alternative the probable subtraction from the community fund of the value of the property under consideration and (probably of more prejudicial though less tangible effect) the impairment of property values arising out of the discouragement to investors?

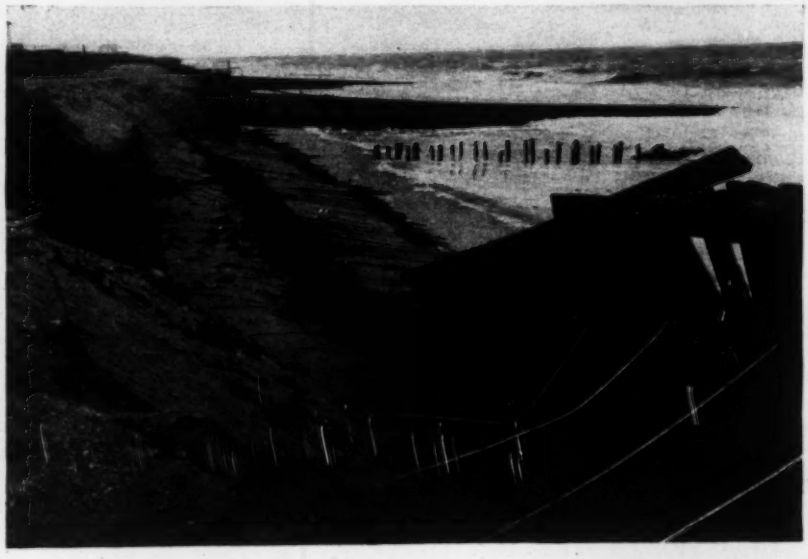
That these losses are preventable has been amply and convincingly demonstrated by experience on the New Jersey coast. The recent storms wrought no damage on the frontages protected by the state-aid jetties with bulkheads or sea walls. Outside the zones of these protective devices serious losses occurred. The line of demarkation between protected and unprotected areas is so sharp that no doubt could remain as to the effectiveness of standard defenses. The results of New Jersey's protective operations have been most gratifying and reassuring and have demonstrated that the cost of protection is much less than the cost of inertia and waste.

Values are both private and public

Are sea-front lands worth to the state and to the communities the cost of their protection? Applying the test of reasonableness, the answer must be emphatically in the affirmative. Certainly there can be no basis for protection entirely at public expense of barren wastes of slight value. The measure of the public's participation should be the public's interest. Economically, the state cannot afford to view with indifference the losses of cities and boroughs which have been transmuted by private labor and capital from worthless sandy wastes into beautiful settlements that contribute by taxation to the support of the state. Attention is invited to the New Jersey coast, and this applies in large measure to all other states from New Hampshire to Florida.

New Jersey has an ocean frontage from Sandy Hook to Cape May of approximately 125 miles. The assessed value of the lands and improvements on

Fig. 3—Sea-wall destruction at Deal Beach by onshore storms of 1932-33. Costly residences border this ocean front.



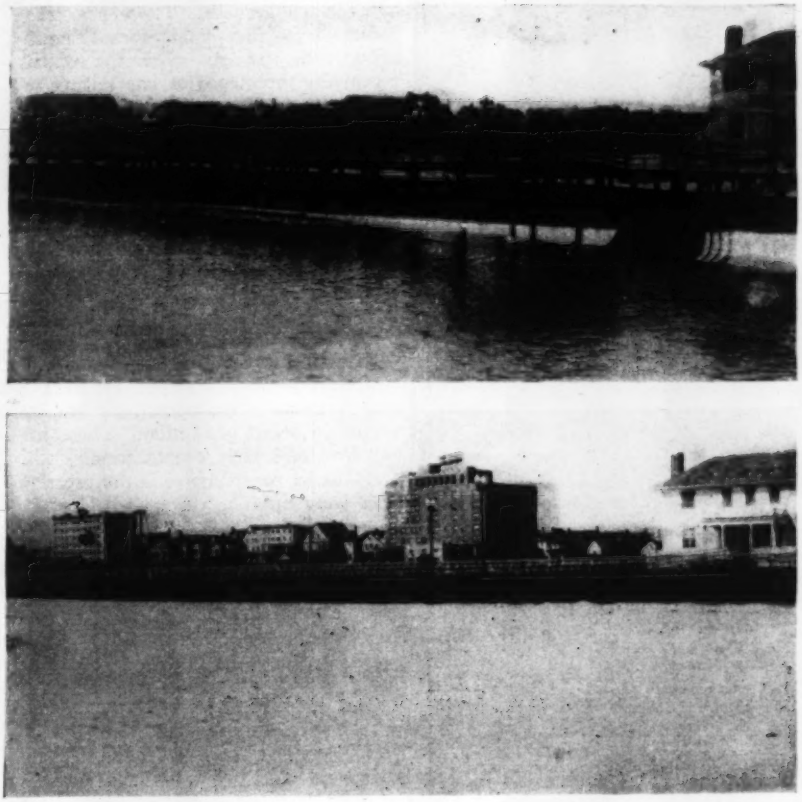


Fig. 4—Section of Asbury Park beach ten years ago and today. Upper view, taken in 1922, shows beach line close to boardwalk and bordering street; virtually no beach is available for bathers, and inshore development is scanty. Lower view, taken in April, 1933, after beach had been built up by sand accretions due to construction of north jetty (Fig. 10). Note inshore development.

which taxes are levied in the municipalities of this ocean frontage, as already stated, amounted in the 1932 tax rolls to more than $550,000,000. A very few years ago this strip of land was a sandy waste of virtually no worth, and today it has no value save that which is created by the attraction of the sea and its accessibility to the cities.

Certainly if a shore hotel or a dwelling is engulfed by the sea, the resulting loss falls immediately on the unfortunate owner. But the entire loss is just as surely passed on to the community at large. Just so much of the wealth by which the municipal government is supported has by this calamity been lost. The community as well as the individual owner is that much poorer. The other property owners of that particular political subdivision must make up the loss by assuming a correspondingly increased burden.

The destruction of a public highway by the sea is immediately recognized as a public loss measurable in financial terms because the road must be restored at a cost which is definitely shown in the state or county or municipal financial statement. The cost of maintenance or repairs is levied upon all the property owners who must as a consequence be subjected to heavier taxation or else dispense with other public services. The destruction of the road is reflected as a direct loss to the community, while the destruction of the dwelling operates as an indirect loss, but the result to the common fund may be approximately the same.

Individual ocean-front owners who enjoy special and exclusive rights should bear an equitable portion of the burden of protection. But in most instances the individual frontage owner cannot effectually protect his own property without aid and guidance from the public authorities, for he cannot compel his neighbors to adopt and maintain protective measures. However elaborate the defenses in front of a given owner's property, they will not avail if the neighbor's wall is breached. The only recourse would be to build cross-walls the entire depth of the property in addition to the front wall, but the expense of this is usually prohibitive, and it is needless if protection be designed according to a general plan.

Has any state established a long-time plan for shore protection? No. New Jersey and Massachusetts have aided in their protection, but in a rather desultory way without any master plans and only as needs became particularly acute and as the afflicted communities could furnish their quotas of funds. New Jersey is apparently on the eve of embarking on a large-scale long-term plan of protection.

Since 1920, when it initiated a policy of state aid, New Jersey has expended $951,000 on state-aid operations through its Board of Commerce and Navigation, in addition to $145,000 on the Barnegat Lighthouse property and $245,000 on the jetties at Manasquan Inlet and the North Shark River jetty. This is exclusive of other appropriations totaling $192,249.64 for state aid to municipalities which have not been met by local contribution.

Physics of beaches

So much for economics and public policy. The engineering of beach protection is based on the physics of beach and wave action, particularly as exemplified by New Jersey beaches but also as characteristic of all beaches.

Beaches are generally classified as barrier and as headland beaches. The headland beach borders the mainland, while the barrier beach is separated from the mainland by an inland waterway generally termed a lagoon. The coast of New Jersey (Fig. 2), which we are considering in detail, has been defined as a winged headland. The headland proper extends from North Long

Fig. 5—Storm waves driving up underneath blew up sections of boardwalk at Atlantic City.

Fig. 6—Atlantic City street that was within reach of the storm waves of 1932-33.

Branch to Bayhead, and the wings are the barrier beaches that extend north to Sandy Hook and south to Cape May where there is another headland.

The headland is characterized by a clearly defined bluff or cliff of varying height, while the barrier beaches are of much lower elevation, much of their area in fact rising very little above the range of the tides. The resistance of the headland to erosion is correspondingly much greater than that of the low-lying barrier beaches.

The coastal plane of New Jersey in past eras has been under the sea. The wings or barrier beaches unquestionably occupy areas that were in recent geological times sea bottom. What formed the barrier beaches, and whence came the earth of which they are composed?

On a sandy seacoast of low elevation, subject in time of extreme storms to overflow, heavy seas breaking continuously and lifting the sand in suspension change the profile of the immediate site and transport the sand thus agitated from its original position to distant points under the influence of hydraulic currents flowing toward the estuaries. The condition will be particularly marked if the general elevation of the land is so low that the sea can rapidly gain upon it because of the absence of walls or cliffs. This condition exists in close proximity to the various inlets, and it is at these points that erosion and accretion have been most marked (Fig. 7). It is the belief that the New Jersey headland may have receded as much as 2 miles, although from measurements the rate of recession is not rapid judged by comparison of surveys during the past 80 years. But assuming that the recession is 1.5 ft. per year, the landward migration would attain highly impressive proportions measured over a period of a thousand years, which is a very short time in the scale of geology. What has become of the earth material washed away from the cliff? Has the action of the waves ground it to impalpable dust or silt? This is doubtful. The theory is that at least a large proportion of this material has been transported to the north to form the Sandy Hook, Sea Bright, Monmouth Beach north wing barrier beach, and the rest transported southward to be incorporated in the south wing barrier beach that extends from Bayhead southward to Cape May.

Another theory is that the barrier beaches were created largely independent of the headland as a source of material, by operation of the waves and largely as erosion from the mainland inshore of the lagoon area. As a matter of fact there is a more or less well-defined cliff formation along much of the border between the solid land and the marshes. Probably therefore the barrier beaches were formed partly of material worn away and transported from the projecting headland and partly of material, whatever its original source, washed up by the waves from the sea bottom.

In view of the immensity of the forces that are at work, it is astounding that there is so little change in the beaches. In fact, there are points on these barrier beaches where stability has persisted for at least 80 years as marked by repeated surveys. There are other points—low-lying land forms newly arisen from the sea—where the sea has repeatedly threatened or has in fact regained possession. There are other points where large acreage of immense value has been regained either by artificial structures or by natural forces. But despite the very considerable change measured from long periods of time the wonderful thing is the fine adjustment relatively in which these forces are held.

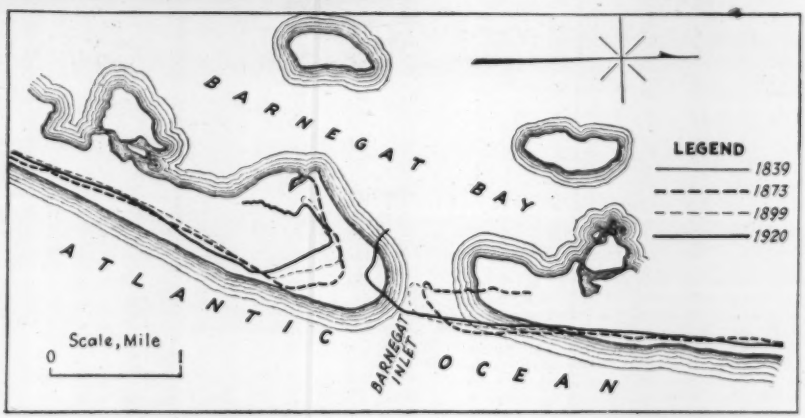


Fig. 7—Changes in shore line at Barnegat Inlet, showing particularly the building out of the north shore and recession of south shore.

Methods of beach protection

Referring to the grouped examples of protective structures shown by Figs. 8 and 9, usually two operations are involved in beach protection. These are bulkhead and jetty construction.

Bulkhead construction is the erection of a watertight barrier parallel to the shore line by which the waves are arrested and prevented from reaching the buildings or roads or whatever structures may be in hazard. This type of barrier is a sea wall, whatever the details of its construction. It may be a simple structure such as an earth dike or a mound of loose rock or a timber or steel fence or a masonry wall; or it may be a structure of more complex details built of two or more of these various materials. It is essentially a dam designed to act as a line of defense against the waves.

The simple dike of rock or earth depends on its mass to resist the wave onslaught. The earth barrier, if subjected to wave washing, will more or less quickly be disintegrated; and the rock, unless confined by cribbing, may settle and scatter. The framed structure of wood or steel depends for stability upon the mass of earth at its rear. The masonry wall, unless of tremendous mass, will also depend in greater or less degree upon the earth support at its rear.

Sea walls are very seldom carefully designed. More often than otherwise

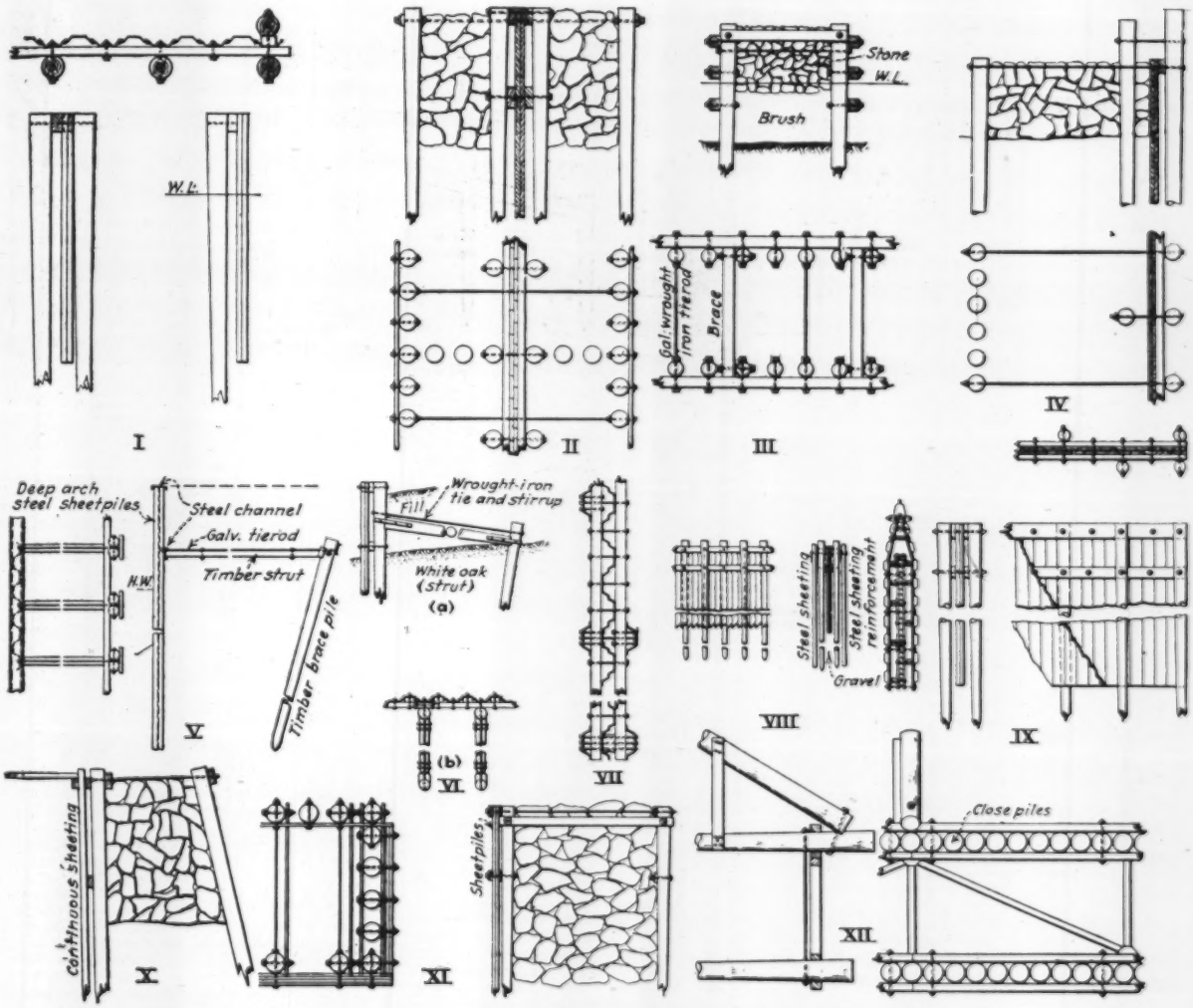


FIG. 8—DETAILS OF BULKHEAD AND JETTY CONSTRUCTION OF TIMBER PILE AND CRIB-WORK, WITH STONE FILL, EMPLOYED FOR BEACH PROTECTION ON NEW JERSEY COAST.

I—A simple groin somewhat similar to IX but containing steel sheetpiling instead of timber sheeting. It is considerably stronger than the light timber sheeting, hence has a greater field of usefulness.

II—A variation of XI, the difference being in the position of the watertight sheeting. The vertical piling at the sides is used to prevent the rock from taking a slope. The volume of rock used is therefore reduced.

III—A type formerly very popular, and effective while the brush and rockfill are maintained. The reason for the brush is merely to save rock. Ultimately, unless fresh brush and rock are frequently added, the rock settles down to below grade.

IV—A serviceable type built at or outshore of the mean-low-water line. It is essentially a breakwater intended to trip the incoming waves and to impound in back (shoreward) of the impervious sheeting the sand carried in suspension in the water that hurdles the barrier. It is effective but not generally applicable as it precludes bathing during most stages of the tide. It is not attractive particularly after it becomes coated with marine growth.

V—A recent type that is serviceable. It is really very similar to types VI *a* and *b* but possesses a superior tieback and brace system.

VI—A familiar type of bulkhead form of sea wall. The construction differs only in that the sheeting is of timber, either tongue and groove or two rows lapped. The steel sheeting, if deep arched, is stronger than the wooden sheeting of dimensions ordinarily employed. The strut timber helps greatly in imparting rigidity, especially if part of backfill is lost for any reason. But the backfill is indispensable for permanence and safety. This type, or variation of it, has, in general, given good service. Unfortunately, in many instances the sheeting is overstressed by the backfill because of faulty design in the structure.

VII—The fence type, using steel master piles as well as steel sheetpiles.

VIII—One method of preserving and reinforcing a timber groin attacked by teredos but still of considerable value. There is seldom any economy in using untreated timber in salt water, except where pollution is so marked that marine life is impossible.

IX—A simple groin of the timber fence type. It is economical and serves very well if employed on a shelving beach. Its usefulness is limited by the depth of water at high tide. It should be constructed of timber treated with creosote or other preservative against decay or marine-borer attack.

X—A type that has served for many years. It is still useful where groins have been constructed and impounded the littoral drift, but where the beach has not been built up with groins it has been topped by waves during recent storms. It is an example of the high desirability of building up a beach in front of a sea wall.

XI—A jetty used in a structure where the water is too deep for the fence type or single-line structure like types I, VII and IX. The only virtue of the rock is to give stability to the timber diaphragm. It is a very useful type but requires careful maintenance as the rock exerts pressure against the sides.

XII—Open-type jetty.

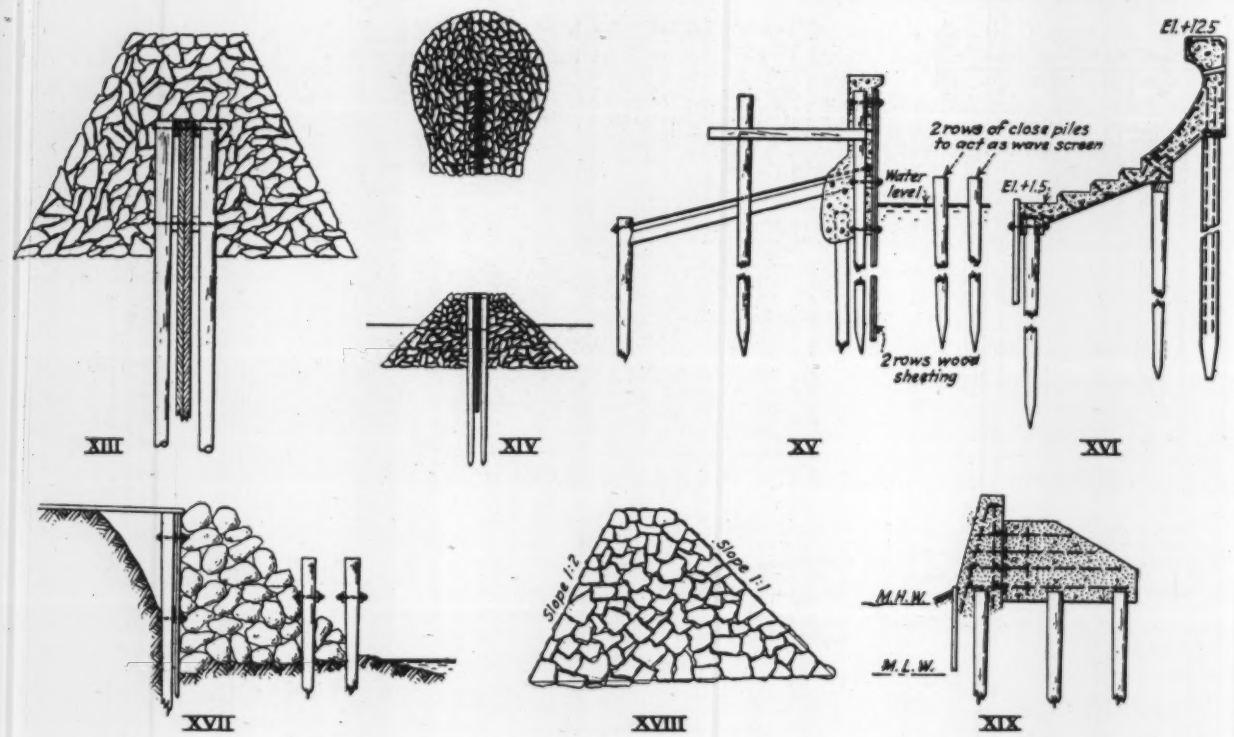


FIG. 9—DETAILS OF TYPES OF SEAWALLS, MOSTLY STONE AND CONCRETE, EMPLOYED FOR BEACH PROTECTION ON THE NEW JERSEY COAST.

XIII—A variation of XVIII. The core is employed to arrest sand, depending upon the rock to break up the waves, which would otherwise destroy the core. The core itself is merely a groin like I, VII and IX. A number of these have been constructed in situations where the depth of water and wave attack require a high degree of resistance. In some instances the core is of steel sheetpiles.

XIV—Similar to XIII with an excess of rock at the outer end. The object is to reduce to a minimum the settlement of rock at offshore end, which should always be expected. It is employed where conditions require a substantial structure yet are not as severe as where type XIII is employed. This illustrates analogy with natural promontories, the offshore ends of which plane away by action of currents and waves.

XV—A rather unusual type. It was heavily stressed by a high fill. Much of it failed during or shortly after construction. Much of it is still standing after 21 years. It is essentially the conventional timber bulkhead weighted down with plain concrete.

XVI—An attractive type. The original plan called for approximately 3,500 lin.ft. Some 420 ft. was demolished in 1920 during construction in a very severe storm. The rest that had been constructed, perhaps 2,000 ft., is still standing and generally in fair condition. The design of toe and the poor horizontal bonding of steps constitute weak elements.

XVII—A serviceable type sometimes employed where the wave attack is too severe for the more simple form of VI *a* and *b*. The object of the two rows of closely spaced piles in front is to break up the waves and to retain the rock in place, as it would otherwise be undermined and dislodged by the waves during storms. It is effective as long as the rock is held in place and the impervious sheeting maintained. But it is costly and often unattractive in appearance. Nevertheless the rock revetment is often indispensable where the earthfill is high.

XVIII—A very simple breakwater or jetty without a watertight core. It is effective if the size of rock is sufficiently varied to reduce voids to a minimum. It illustrates the reason for the crib type with closely spaced rows of piles at the side, like VI and XI. Obviously, the slopes of the rock section make up a very large volume. The rock will not stand on a 1-to-1 slope, but will tend to settle and roll until a good foundation of rock settling into the sand is obtained. The slope will probably range around 1 vertical to 1½ horizontal, or 1 vertical to 2 horizontal, depending on rate at which shore builds up.

XIX—This type of fine appearance failed completely. No plans are available, this plan being reconstructed as well as possible from field observations. It is not known just how long it endured. But the cutoff sheeting system is very poor. Shearing at the joint between main wall and counterfort is marked.

they are merely constructed without much regard to the stresses to which they will be subjected. Admitting the meagerness of quantitative knowledge of wave impact, there is scanty excuse for building sea walls or bulkheads of dimensions obviously far inferior to the requirements that the wall must meet as a retaining wall with respect to the earth which it impounds. A timber bulkhead of 4-in. tongue-and-groove sheeting, with master piles and stringers of corresponding dimensions, or a very light steel bulkhead might serve very well in selected situations for a bank of earth 5 or 6 ft. high. But to extend the same plan to a site where the bank is 18 or 20 ft. high is to invite destruction, as at Deal Beach (Fig. 3).

This sort of thing is observed everywhere. The explanation is that owners and engineers are too often forced to gamble on the possibility that the walls will not be heavily attacked. Financial limitations control.

The defense barrier should meet at least two tests: (1) it must have a firm foundation on bedrock or other consolidated stratum or on piles; (2) it must have sufficient strength or mass to retain a backfill of wet earth.

Quantitative information on earth pressures is meager, and dependable data on the force of waves impinging on wall face are still more so. Assumptions have to be made, based on experience and observation. But this statement can be safely offered as a guiding rule:

As long as the earth backfill remains in place in back of and supporting the sea wall, protection against the waves will be attained. This is not to play with words, but is stated in order to emphasize that upon the backfill's integrity rests the security of the sea wall and that sound design relies upon the support of the backfill. If the wall is so frail that it is deformed and weakened by outward pressure exerted by the backfill, it will surely fail as a sea wall.

The second element in the sea-defense scheme is a system of groins or jetties (Fig. 10) which extend each

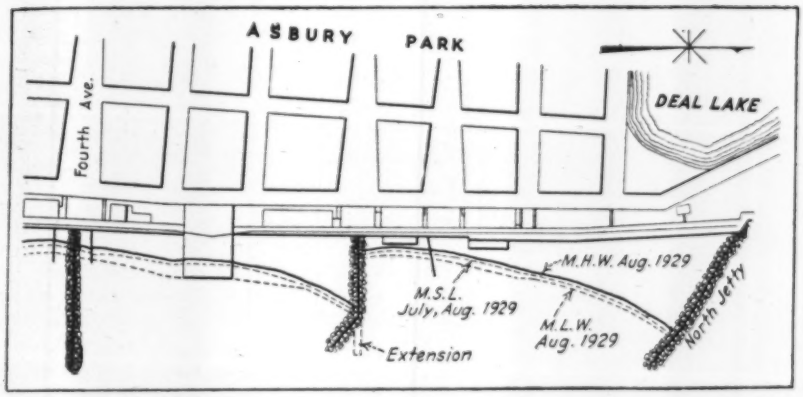


Fig. 10—Section of Asbury Park ocean front, showing action of state-built jetties in building up beach. See also Figs. 2 and 4.

in a continuous line seaward more or less at right angles to the direction of the sea wall. Like the sea wall, they may be built of any of the standard materials of construction utilized in cofferdams, retaining walls, etc. Frequently the designs of sea wall and groin are closely similar to each other. The object of the jetties or groins is to arrest and impound sand in motion along the beach. They act like promontories with the bays between where sand set in motion by waves comes to rest. More often than otherwise a comprehensive plan requires a continuous sea wall firmly joined to a system of jetties or groins.

The argument that the public should provide sea defenses where the attack is serious is that public property will be in hazard if private owners are overcome. In many situations roads are now in acute danger, the private holdings having been wiped out. In typical subdivisions the lands are so laid out that between the main highway and the ocean there is a strip of land privately owned. As the sea works landward (and that is almost universally the case) the private owners are at great expense to protect their lands. In protecting their frontage they are necessarily protecting the public highway and the owners inshore of the highway. Seldom, if ever, does the public authority give any aid, while it enjoys this immunity from expense through the seafront-owners' efforts. The practice is perhaps everywhere to wait until the shore owner is utterly defeated and the road partly or wholly destroyed. Then the highway authority devises structures which are carefully located within the highway boundaries. In addition to condemning utterly the private holdings to the devil and the deep sea, this system yields inferior protection because the waves beating against the road wall cast huge volumes of water loaded with sand and driftwood upon the highway. The same sea wall, if constructed 150 ft. seaward of the highway, would serve far more effectively as protection. At the same time it would preserve to the tax roll all the valuable private property.

In New Jersey a movement has been launched to provide from state funds a schedule of modest appropriations to support a program of beach protection. Companion bills in the legislature have for their object the allocation to this use of small proportions of the receipts from the gasoline tax. Some opposition will be offered, notably from automobile and highway interests. This resistance has no sound basis, for the automobilist should welcome the preservation of intensively used primary highways.

This opposition, however, should be overcome. It is not enlightened opposition. It is based on the fear that the application of these funds to shore protection will reduce *pro tanto* the yield available for county roadbuilding. But it will not, in truth, so operate for the object is very largely to conserve existing highways along the beaches. Of these important highways many miles are now in gravest peril in every county on the ocean front. The object is largely the conservation of existing highways, not diversion of funds to non-highway purposes. These roads can be saved, and the loss that would result from destruction of private holdings can be averted. If the roads are destroyed by the sea, they will be rebuilt at public expense anyway, and the county road aid will be immediately and severely affected. Surely it is at least as sound economics to save an existing road as supinely to await its destruction and then rebuild the road at necessarily increased cost; and then in addition inevitably bear the increased cost of sea defenses.

It is high time for recognition of the immense value of the seacoast to the states and to the country at large and for planning their preservation and utilization to the best interest of the people. To quote the words of that eminent publicist and economist, the late Commissioner William Lawrence Saunders, of the New Jersey board of commerce and navigation, "The ocean boundary is a band of gold and must be conserved."

Specifications Suggested for Asphalt Bridge Plank

Extensive series of tests on samples from seven manufacturers coupled with service records form basis for proposed specifications

By L. H. Koenitzer
Applied Mechanics Department, Kansas State College, Manhattan, Kan.

SINCE 1925 several million feet of asphalt plank have been laid on more than 1,000 bridges. Although exact figures are not available, the demand for such plank has increased rapidly during the past two years. In spite of this wide use, there is a very great lack of knowledge in regard to specifications and tests for asphalt bridge plank. Manufacturers' specifications naturally differ, but at present they are the only guide that the purchaser has. The data reported in this article are based on an investigation involving practically every test suggested in the specifications of seven manufacturers and of eighteen cities and state highway departments. The tests were made on five samples of plank of various thicknesses submitted by the manufacturers since Jan. 1, 1933, and on two other brands of plank furnished in July, 1932. A test was also run on a brand of plank that is giving excellent service after 2½ years of use. In all, seventeen samples submitted by seven manufacturers were tested. In carrying out the tests the writer was aided by suggestions made by Prof. C. H. Scholer, consulting materials engineer of the Kansas highway department and head of the applied mechanics department of Kansas State College, who is chairman of an A.S.T.M. committee appointed to look into the possibilities of a specification for asphalt bridge plank.

Asphalt bridge plank is the outgrowth of work done on a prepared rail filler that consisted of asphalt, fiber (felt) and aggregate extruded through a die in the proper shape to fit between the rail and the adjacent pavement. The engineer in charge of this product conceived the idea of making plank for use as a surfacing material between the

TABLE I — TEST RESULTS ON ASPHALT BRIDGE PLANK

Thickness of Plank	2 In.	1½ In.					1¼ In.		1 In.					¾ In.	½ In.	
Brand	D	A	B	D	E	F	G	B	D	B	C	E	F	F Stone	C	C	F
Specific gravity 70 deg. F.	1.42	1.41	1.57	1.41	1.59	1.44	1.56	1.68	1.39	1.55	1.83	1.76	1.50	1.52	1.81	1.84	1.41
Absorption 70 deg. F., per cent	.6	.7	.6	.7	.6	.3	.8	.3	.5	.4	.3	.6	.7	.6	.3	.2	.9
Bitumen, per cent	37	45	39	44	38	40	38	30	50	29	36	33	37	33	37	32	41
Felt, per cent	16	21	19	18	6	16	15	4	19	24	25	6	14	14	25	11	14
Ash and mineral, per cent	47	34	42	38	56	44	47	66	31	47	39	61	49	53	38	57	45
Compression shear 77 deg. F.																	
350 lb./sq.in., 16 min	.24	.28	.18	.20	.33	.20	.19	.08	.20	.12	.08	.13	.13	.19	.05	.04	.06
700 lb./sq.in., 10 min	.99	.90	.37	.84	.99	.54	.97*	.15	.86	.26	.13	.67	.36	.49	.10	.07	.11
Compression shear, 135 deg. F.																	
100 lb./sq.in., 5 min	.18	.27	.18	.22	.39	.40	.11	.07	.24	.11	.28	.19	.15	.23	.11	.05	.05
350 lb./sq.in., 5 min	.95	.76	.95	.93*	.97*	.88*	.78	.26	.97*	.46	.64	.80	.63	.82	.30	.16	.14
Max. fiber stress																	
32 deg. F., lb./sq. in	...	843	1,315	1,152	787	1,237	827	1,441	1,174	1,720	1,374	1,191	1,316	1,298	1,712	2,084	1,382
Deflection, 32 deg. F., in	...	.20	.14	.13	.18	.14	.17	.08	.13	.11	.32	.09	.09	.16	.08	.21	.08
Increase in volume, 200 deg. F.																	
20 hours, per cent	1	14	6	10	16	11	..	41	9	4	16	14	9	6	7	5	9
Impact, 77 deg. F.																	
Loss in ft.-lb.	...	45	28	52	40	26	56	24	34	20	20	18	21	36	10	10	9
Impact, 32 deg. F.																	
Loss in ft.-lb.	...	16	18	...	11	17	15	19	...	14	25	11	19	30	8	9	6
Penetration test, 135 deg. F., mm.	1.0	.6	1.0	1.1	1.7	1.7	1.1	1.6	1.3	1.0	2.0	1.4	1.2	1.1	1.2	1.0	1.5
Brittleness test, 32 deg. F., per cent passing	100	100	100	75	0	50	50	50	75	0	100	0	0	75	50	0	100

* Did not take the load.

rails, and these plank, also extruded through a die, were used on two railroad crossings in 1925. The following year changes were made in the composition of the plank, and it was laid on a dozen or more bridges in 1926. It was found that some of the earlier installations did not give satisfactory service as the material had a tendency to roll under traffic. Since 1927 at least seven manufacturers have been experimenting with various mixes and materials, until at the present time most manufacturers are able to turn out a product that will give fairly satisfactory service.

Before making a study of the physical characteristics of asphalt plank in the laboratory it was thought best to get the experience of engineers over the country. A questionnaire was sent to each highway department and to the engineers of fourteen large cities. Eight replies were received giving information on asphalt bridge plank already in service. From these replies it was obvious that tests made on asphalt bridge plank should measure its ability to withstand loads under various temperature conditions and its toughness or ability to withstand shock, particularly when the plank is laid on an uneven sub-base. The replies indicated further that the proper percentage of asphalt, felt and mineral aggregate should be determined. The design of an asphalt plank is very similar to the design of an asphaltic top except that the felt is added so that each plank will act as a unit, without regard to the adjacent sections.

Only eighteen specifications on asphalt bridge plank were received out of 47 replies from state highway departments and city engineers. Most of these specifications were copies from the manufacturers' specifications.

Manufacturers have used various methods and equipment to study the properties of asphalt bridge plank, but in the final analysis they all attempt to measure the toughness, ability to withstand shear, penetration, quantity of bitumen, per cent of mineral aggregate and the per cent of felt in the plank.

For the purposes of this investigation, the following tests were made on each sample: (1) composition including asphalt content, felt content and mineral aggregate; (2) loss on heating to 200 deg. F.; (3) specific gravity; (4) penetration; (5) absorption; (6) impact; (7) compression shear; (8) transverse strength; and (9) brittleness.

Table I is a tabulation of the results, and Table II is a list of suggested specification limits.

Description of tests

Considerable difficulty was experienced in cutting out samples from asphalt bridge plank by hand. After some experimentation it was found that by placing an ordinary 8-in. circular saw blade on the shaft of a milling machine, it was possible to cut out specimens that were very uniform in size and shape. The machine was run at a low speed and the plank was pushed by hand under the blade at the rate of about 15 ft. per minute. In cutting out a specimen from 1½-in. plank it was necessary to run the plank through twice.

Composition—In determining the per cent of bitumen several methods were used, including the method recommended by A.S.T.M. D4-27 and extraction, using a Soxhlet Extractor and a Rotarex machine. It was found that the test was best performed by weighing out a representative 100-gram sample, soaking the sample in carbon tetrachloride for two hours and extracting by placing in a Rotarex machine. One sample was sufficient for the extraction test, as duplicate results on the same brand of plank checked within 0.5 per cent of the average of five tests; the greatest variation between individual samples was less than one per cent. Using the Soxhlet method proved slow and tedious. Carbon tetrachloride, used as solvent, gave practically the same results as carbon disulphide, with the added advantage that it was fire-safe. Suggested specification limits would be 30-50 per cent bitumen.

Fiber—In the determination of the percentage of felt there is a possibility of an error of as much as 50 per cent, depending upon the kind of mineral aggregate contained in the sample and the temperature at which the felt is burned. If the sample contains limestone dust and the felt is burned off at a high temperature, a considerable amount of carbon dioxide will be driven off which in the past has been considered as part of the felt because specifications have defined the per cent of felt as the difference in weight before and after burning, divided by the original weight of the extracted sample. To check the existence of this error, samples were recarbonated by adding ammonium carbonate to the ash and driving the ammonium off at a low temperature. A suggested specification on felt is from 5-25 per cent, after correction for loss of carbon dioxide has been made by recarbonization.

Mineral Aggregate — The material remaining after the felt has been burned off consists of ash and mineral aggregate. The aggregate should consist of roofing slate, rock dust and fine crushed rock. Some asbestos fiber was noted in one or two samples tested. A suggested specification on mineral aggregate would be 30-50 per cent of roofing slate, rock dust, crushed rock or inorganic material.

Loss on Heating—Samples weighing 150 grams were placed in an oven maintained at 200 deg. F. for a period of 20 hours. It was found that the loss on heating varied from 0.45 to 3.02 per cent. There is no indication that the loss on heating should be made a standard test.

Specific Gravity — The specific gravity was made on 2x6-in. samples that had been soaked in water for 24 hours. The specific gravity of each sample varied with its asphalt, felt and aggregate content. A more significant test involved the determination of the specific gravity of the specimens before and after heating at 200 deg. F. for 20 hours. The change in specific gravity was an indication of the bulking effect caused by heating.

Absorption—An absorption test was made on 2x6-in. specimens that were immersed in water at 70 deg. F. for 24 hours. The percentage of absorption

varied from 0.17 to 0.95 per cent. A maximum specification of 1.0 per cent would be desirable.

Penetration — Various specifications suggest a surface-penetration test, but after some experimenting it is the writer's belief that such tests are not satisfactory, as plank that has been stored for several years will have a low penetration. Such a test is not recommended.

Impact—Samples for making impact tests were 6x6 in. for plank 1 in. or less in thickness, 6x4 in. for 1¼-in. plank, 6x3 in. for 1½-in. plank and 6x2 in. for 2-in. plank. The test was made by placing a sample in a pendulum impact machine with a 60-ft.-lb. capacity which may be purchased or made in a shop in a very short time. The planks were placed in the machine so that the grain and the 6-in. dimension were parallel to the vertical axis of the pendulum. The loss in energy, recorded in ft-lb., was considered the ability of the plank to absorb shock or impact. Before testing the specimens they were placed in water maintained at 77 deg. F. for five hours. The results showed a loss of 26 to 52 ft.-lb. of energy on the 1½-in. plank. On the 1-in. plank the impact loss varied from 18 to 35 ft.-lb. Each value is the average of five specimens from the same brand of plank. From observations in the field, plank with the ability to absorb impact the best is giving the best service. It was noted in making the tests that 1-in. thick F brand plank, with stone crushed into the surface, had an impact value of 35 ft.-lb. as compared to 21 ft.-lb. for the same size and brand without the special surface.

In making impact tests at 32 deg. F. it was found that certain brands of plank had only about one-third the impact value that they did at 77 deg. F., while other brands had practically the same impact value at both temperatures. A suggested specification would be that plank when tested at 77 deg. F. and as described in this article should have a minimum loss on a 60-ft.-lb. impact machine of 30 ft.-lb. for 1¼- and 1½-in. plank, and that for 1-in. plank the impact loss should be at least 20 ft.-lb. If the plank is to be used in the northern part of the United States, the impact loss at 32 deg. F. should be at least 50 per cent of the impact loss at 77 deg. F.

Compression-shear — The compression-shear test was designed to measure the ability of plank to withstand concentrated loads and to determine the softness of the plank. Specimens of 6x6-in. size were placed in a constant-temperature bath maintained at 77 or 135 deg. F. for a period of five hours, after which they were placed in a transfer dish containing water at the prescribed temperature. A steel plunger 2x2-in. in cross-section and 6 in. long was placed in the center of the specimen, which was mounted in an ordinary screw-type compression machine. A total load of 1,400 lb. was maintained on the specimen (temperature, 77 deg. F.) for 10 min. A dial reading to 1/1000 of an inch, with a capacity of 1-in. displacement, was placed against the compression head of the machine. An initial reading was taken on the dial when there was a total load of 100 lb. on the 2x2-in. plunger. Another reading was taken after a total load of 1,400 lb. had been maintained on the specimen for 10 min. The load was then increased, and a total load of 2,800 lb. was maintained for another 10 min., after which time the dial was read again. The range for all thicknesses and brands of plank was 0.077 to 0.328 in. for a total load of 1,400 lb. and 0.130 to 0.991 in. for a total load of 2,800 lb.

Similar tests were made at 135 deg. F., with a total load of 400 lb. maintained for 5 min., which gave a range from 0.071 to 0.396 in.; and with a total load of 1,200 lb. for 5 min., which gave a range of 0.263 to more than 1 in.

The deformation for all thicknesses of plank was practically the same for the same temperature and loading. The deformation of the plank at 135 deg. F., with a total load of 400 lb. for 5 min., gave practically the same deformation as when the plank was tested at 77 deg. F. with a total load of 1,400 lb. The same relationship does not occur between the specimens tested at 77 deg. F., total load 2,800 lb., and specimens tested at 135 deg. F. with a total load of 1,400 lb. At the higher temperature the deformation is considerably greater in spite of the fact that the load and the time of loading are less than half of what they are at the lower temperature.

A suggested specification would be a minimum deformation of 0.2 in. at 135 deg. F. when a total load of 400 lb. is applied for 5 min., and a maximum deformation of 1.0 in. at 135 deg. F. when a total load of 1,400 lb. is applied for 5 min. It is not considered necessary to determine the deformation at both 77 and 135 deg. F.

Transverse Test—Specimens 2x6 in. in size were tested as a beam in flexure over a span equal to three times their thickness. The load was applied at the center of the span by a ⅜-in. steel rod. The deflection of the beam was measured by placing a dial reading to 1/1000 in. on the underneath side at the center of the span. Since the span was varied with the thickness of the specimen, it was found that the maximum fiber stress varied considerably for different thicknesses of plank, namely from 787 to 2,084 lb. per sq.in.; the deflections varied from 0.08 to 0.32 in. A suggested specification would be a minimum fiber stress of 1,000 lb. per sq.in., with a minimum deflection of 0.12 in.

TABLE II — SUGGESTED ASPHALT BRIDGE PLANK SPECIFICATIONS

Test	Proposed Limits
Asphalt, per cent	30–50
Felt, per cent	5–25
Ash and mineral aggregate, per cent	30–50
Absorption, maximum per cent	1
Impact (77 deg. F.), 1¼- to 1½-in. plank, minimum, ft.-lb. loss	30
Impact (77 deg. F.), 1-in. plank, minimum, ft.-lb. loss	20
Impact (32 deg. F.), minimum, 50 per cent of impact at 77 deg. F.	
Compression shear 135 deg. F.	
100 lb. per sq.in., 5 min., minimum, in.	0.2
350 lb. per sq.in., 5 min., maximum, in.	1.0
Transverse test (32 deg. F.) minimum fiber stress, lb. per sq.in.	1,000
Deflection, minimum (32 deg. F.), in.	0.12

Brittleness Test—Perhaps no single test is so important as the brittleness test. If it is impossible to fasten the plank to the base wihout breaking or splitting it, the plank is not adaptable to field use. Asphalt bridge plank should not be permitted to be nailed to a wooden plank floor when the temperature is below 40 deg. F. On one occasion the writer recalls the placing of an asphalt bridge plank floor during freezing weather. The planks were heated by placing them in hot water. After removing, the planks were wiped dry and then nailed to the sub-base. Less than one year after placing the plank, a survey showed that about 50 per cent of the plank was split from the nails to the edge of the plank (longitudinally).

The brittleness test was made by dropping a 14-lb. hammer on the head of a 30d nail, placed 2 in. from the edge of a 6x6-in. specimen. The height the hammer was dropped was sufficient to drive the nail through the plank with one drop. If four nails, one near each corner, did not split or break off the plank, it was reported as passing the test 100 per cent. If one corner failed, 75 per cent passing was reported, and if all four corners failed, the plank was reported as 0 per cent passing. Similar tests were performed by driving the nails by hand; also by driving the nails through in several blows; these were not successful. The above tests were made on specimens that had been maintained at 32 deg. F. for five hours. A specification should require 100 per cent passing.

Summary

The information available indicates that manufacturers have been continually changing their products. So frequent have been these changes in the past that it has been almost impossible to get a plank floor of the same characteristics on two different bridges in the same state in one construction season, even though all the material came from the same manufacturer. It appears that manufacturers have changed from one extreme to another in hopes of satisfying the consumer, and the changes have not always been in the direction of improvement. Perhaps the consumer is at fault by not having had definite specifications for manufacturers to meet. A fixed specification would have prevented the change from hard to soft asphalt plank, which has occurred in the past three years. It is hoped that the suggestions in this article may prove of value in formulating a satisfactory purchase specification.

Cliff-Dwellers of 1000 A.D. as Underpinning Experts

What the inhabitants of Pueblo Bonito did to prevent destruction of their city by a great rock—Their footing walls were probably useless as supports, but valuable in retarding wind erosion

By J. B. Hamilton and Frank A. Kittredge
Assistant Engineer and Chief Engineer, Respectively, National Park Service

A PRIMITIVE engineering effort by early American cliff-dwellers to brace up a detached cliff section weighing 30,000 tons, which appeared to threaten their pueblo, is a special feature of interest in the Chaco Canyon National Monument. Although an analysis shows that the rock is stable and that the masonry wall would have been entirely ineffective in stopping overturning, it is evident that the ancient masonry brace was effective in protecting the base of the rock from serious wind erosion. The modern engineering plan to protect the rock involves the same general procedure used by the ancient craftsmen, as it provides a masonry wall to stop wind action, supplemented by the grouting of the horizontal cracks at the base to provide more bearing area in front of the gravity axis.

Behind the Pueblo Bonito ruins in Chaco Canyon, N. M., is a detached fragment of cliff 100 ft. high, 140 ft. wide and more than 30 ft. thick, which is seriously undercut along the base and has the general appearance of instability. Apparently this same condition existed when the pueblo, consisting of about 400 rooms, was inhabited by 1,200 Bonitans. Concerned over the threatening rock, which literally stood at their back wall, the people of the city undertook to prevent the possible fall of the rock by building masonry braces under the undercut section. This work was done more than 900 years ago; the date is conclusively established by tree-ring measurements on a timber post found embedded in the masonry. It is known that in about the same period the pueblo was deserted.

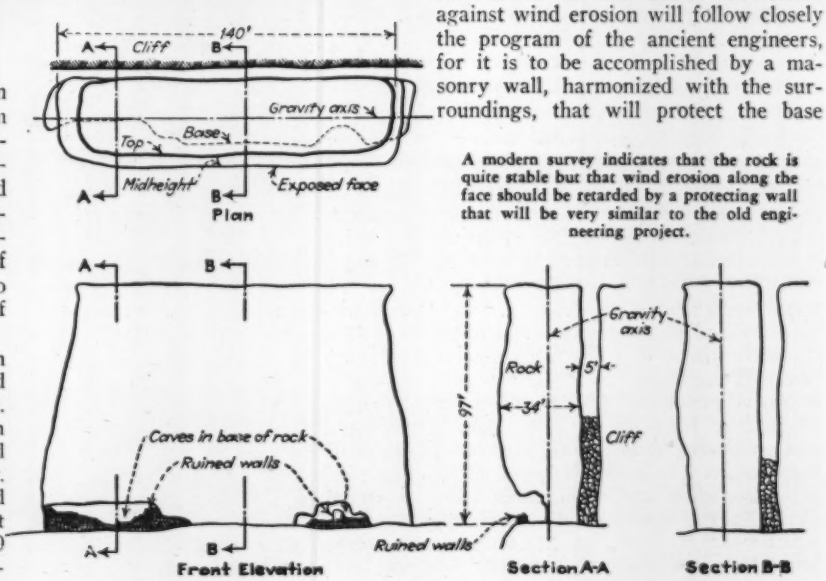
Today, two extensive portions of these old walls still survive. They indicate that about 200 cu.yd. of masonry must have been contained in the original protection work. The Indians of today apparently appreciate the purpose of the ancient masonry, for the modern Navajo name for this locality is translated "Place-of-the-braced-up-cliff."

This threatening 30,000-ton rock caused the inhabitants in Pueblo Bonito 900 years ago to attempt to brace it up with masonry along the front.

Almost 200 cu.yd. of ancient masonry attests the engineering efforts of these early Americans to stabilize this cliff fragment.

It is probable that the masonry would not have been able to hold the rock against overturning if the natural base had actually eroded to the point of instability. However, the wall has proved effective (and probably essential) in stopping the serious wind erosion and the action of moisture and frost.

A recent survey was made to determine the stability of the rock and to suggest a program for its protection. Although undercut extensively, the rock is in stable equilibrium, and there is no danger of its falling except perhaps in case of earthquake. The present problem is to provide for stopping wind erosion and to decrease as much as practicable the thrust of frost action in the debris at the rear. Protection against wind erosion will follow closely the program of the ancient engineers, for it is to be accomplished by a masonry wall, harmonized with the surroundings, that will protect the base



A modern survey indicates that the rock is quite stable but that wind erosion along the face should be retarded by a protecting wall that will be very similar to the old engineering project.

from erosion and keep out moisture.

In addition, a modern engineering tool will be used—in the form of grout applied to the horizontal cracks near the base—to increase the bearing area. The other feature of the suggested program will be to provide a waterproof coating over the debris between the rock and cliff to keep out moisture and decrease the thrust of frost action.

All activities in the national parks are carried out under the general direction of Horace M. Albright, director of the National Park Service. Frank Pinkley is in active charge of all the national monuments in the Southwest, and Hurst R. Julian is custodian of the Chaco Canyon National Monument.

Rolling a 400-Ton Railway Span Into Position From Falsework

By George B. Taylor
Chief Engineer,
The Berlin Construction Co., Inc., Berlin, Conn.

A NEW plate-girder span, weighing 400 tons, carrying the tracks of the New York, New Haven & Hartford Railroad over Connecticut state highway No. 108, at Buckland, Conn., was successfully rolled into position on the night of Sept. 18 and 19, 1932, after having been erected complete on falsework adjacent to the old span, which had to be removed as a part of a highway-widening program.

The bridge is of the half-through type, double-track, 50-ft. clear span and slightly skewed. The center girder weighs 25 tons, and the two side girders weigh 17 tons each. A solid concrete floor resting on steel cross-beams 2 ft. on centers is used. One new abutment was built, and the tracks were carried on a temporary wood trestle between the new and intermediate old abutment during construction.

The new span was built on falsework immediately south of the existing trackage. The girders were transferred from the cars to the falsework by a locomotive crane and then skidded out to position. The steel was assembled and riveted, the concrete slab was poured, and waterproofing, ballast and ties were placed in position.

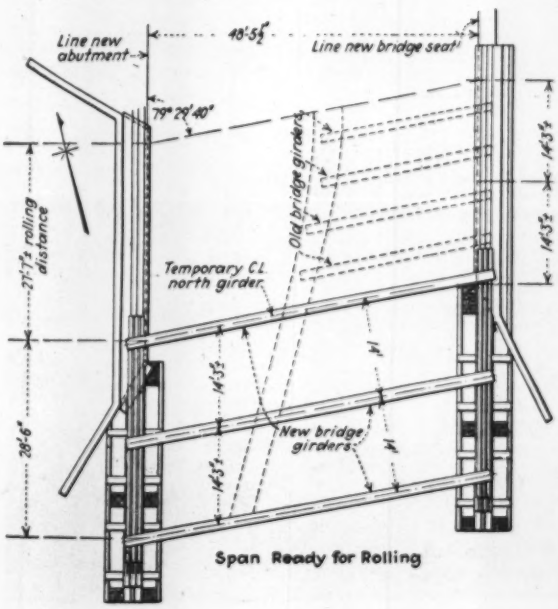
After the last train crossed the old structure Sunday evening at 7:30, the tracks were removed, the old steel span was swung out, the temporary trestle demolished, and the interfering portions of the old abutment which were standing in the center of the new highway were torn down.

The rolling in was started at 9 p.m. The moving structure was supported at each abutment by a grillage of three 15-in. I-beams 35 ft. long, continuous under the ends of the main girders, bearing on 3-in. rollers spaced 1 ft. on centers, which rested on three lines of 107-lb. rails on the falsework and abutments. Two double-drum hoisting engines, with $\frac{5}{8}$-in. cable reeved through triple blocks giving a seven-part line, were used to move the span, which was in place ready for the lowering operation at 10 p.m.

Since the structure was rolled in 13 in. above its final bearing, it was necessary to lower it to position on the bridge seats. It had been impossible to place jacking bents in advance, as the space between the old abutments was in use as a highway, and the space between the abandoned abutment and the new one was occupied by the temporary railroad trestle. Accordingly, steel jacking bents were suspended from the structure and rolled into place with it. Continuous rafts of old ties were then laid down adjacent to the abutments, and a cribbing of 12x12-in. timbers was placed on them to distribute the loads from the four 100-ton and four 50-ton ratchet-jacks that were used. After the rolling grillage, rollers and rails were removed at each abutment and the cast-steel pedestals placed in position, the span was lowered, two jack settings being necessary. Solid bearing at one end or the other was maintained throughout the operation.

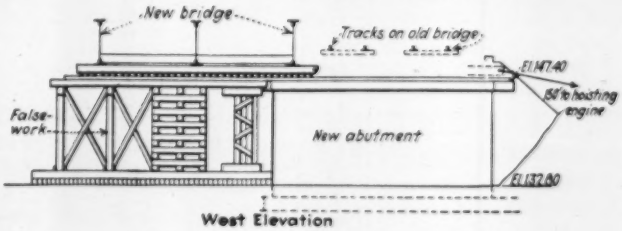
Lowering was completed at 6:20 a.m., and the railroad company's forces carried the trackage over the new structure by 7:30 a.m. The first train passed over the bridge at 8:35 a.m. Monday, thirteen hours after the work was begun.

The bridge was constructed for the New York, New Haven & Hartford Railroad Co., under the supervision of E. E. Oviatt, chief engineer, and F. J. Pitcher, engineer of structures. The general contractor was the Lathrop & Shea Co., New Haven, Conn. The subcontractor for the structural steel, which included the temporary support of the structure and the rolling and placing of it in its final position, was the Berlin Construction Co., Inc., of Berlin, Conn., and the operations described were carried out by Samuel A. Johnson, superintendent of erection.

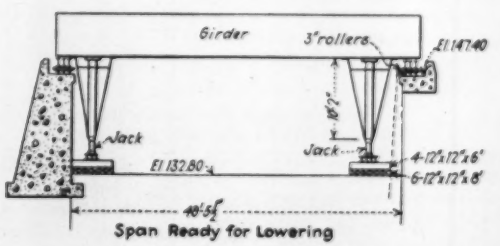


Span Ready for Rolling

Falsework and grillage arrangement used in operation of rolling into position a new double-track railway girder span.



West Elevation



Span Ready for Lowering

Handling 2,000,000 Cu. Yd. of Excavation at Boulder Dam

Rock from canyon walls and riverbed gravel moved out by truck fleet and trains—Cofferdam construction coordinated with excavating program—Daily peak reaches 7,200 truckloads, or about 38,000 cu.yd.

EXCAVATION of canyon walls and riverbed, combined with the building of the two permanent cofferdams, representing the handling of more than 2,000,000 cu.yd. of materials, have provided the major activity on the Boulder Dam project during the past months. In the five months following river diversion (Nov. 13) this work resulted in completing both cofferdams, preparing the canyon walls down to El. 655 (10 ft. above water surface) and removing more than 800,000 cu.yd. of gravel from the riverbed. Most essential, from the standpoint of avoiding interrupted work and delay, was the construction of the upstream cofferdam, and more than 400,000 yd. of earthfill was placed during December. The rate of excavation reached a high peak during February and March, and on some days as many as 7,200 truckloads of material, or about 38,000 cu.yd., were removed from the canyon in three shifts. This excavation program represents one of the major construction operations on the project and, although involving no distinct innovation in equipment or procedure, constitutes an important chapter in the building of Boulder Dam. The features of this excavation program, including the various types of work, methods used and some of the yardage statistics, are presented in the following.

Before the river was diverted from the canyon on Nov. 13, 1932, excavation work, with the exception of the spillways on the rim of the canyon, had been confined to preliminary scaling of loose rock along the canyon walls, cuts for the intake towers and the beginning of work in the upper cofferdam site. These operations had been given secondary consideration as compared to the more essential work of driving and lining the diversion tunnels, which was the key to the progress on the project as a whole, prior to diversion. With the river out of the canyon, it was possible to concentrate on the excavation of the entire canyon area, and this work, in turn, became the main feature of construction, as its completion was essential for the beginning of actual dam construction (Fig. 1). Combined with this excavation program was the construction of the two permanent cofferdams, which required the transportation of earthfill into the canyon. The correlating of this transportation program was an important feature of the speed of operations.

Equipment

No new major units of equipment were required for the large-scale river excavation program, with the exception of the usual replacements in the truck fleet. Excavating equipment used in the river and in the borrowpit for the earthfill included the 2½-yd. electric shovels equipped with 3½-yd. buckets used to excavate the diversion tunnels and one 5-yd. electric dragline that had been used at the gravel deposit.

Truck operation during this excavation work, as well as from the beginning of the job, is of interest because of the size of the fleet and the output under conditions of exceptionally hard usage. During the peak of riverbed excavation the main fleet consisted of 70 to 80 trucks operating during three shifts. These trucks were used exclusively to transport material from the shovels loading in the canyon to the docks, where the material was transferred to trains for removal, and did not include those used elsewhere on the work. Ordinary maintenance and repair operations were carried on in an extensive shop near the low-level mixing plant, later moved to the wide crest of the upper cofferdam as soon as it had been completed. Size of the units varied all the way from small trucks with bodies carrying average loads of 3 to

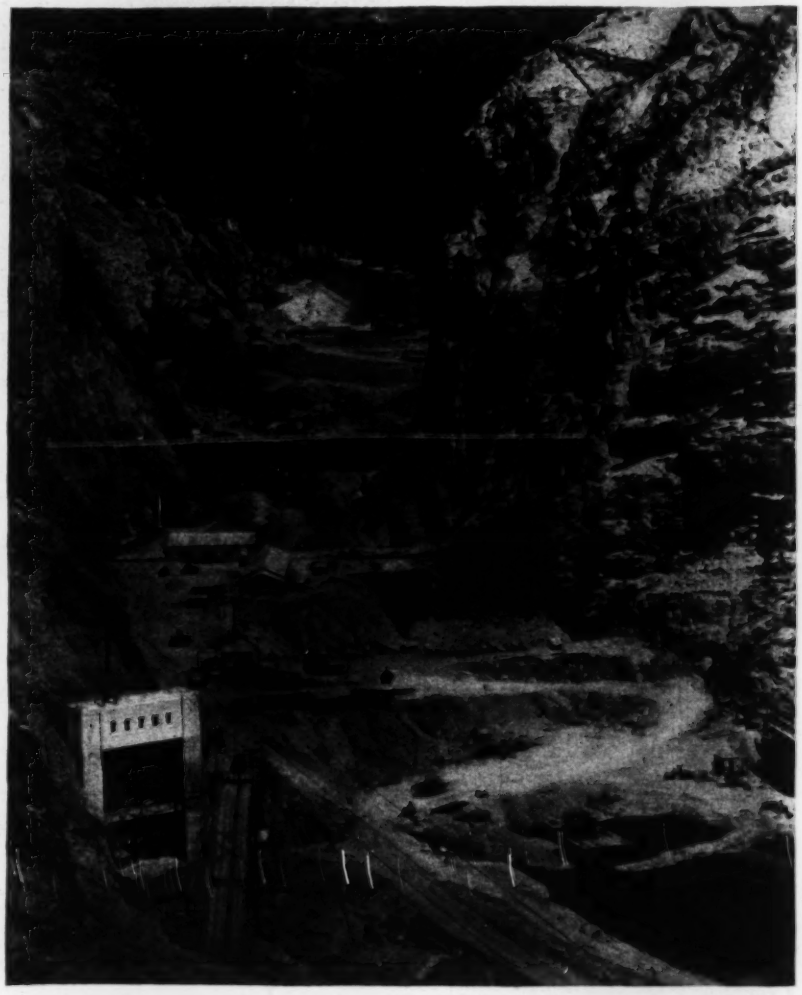


Fig. 1—Looking up Black Canyon, with Boulder Dam excavation operations in full swing. Lower rock barrier and cofferdam in foreground. Next above can be seen benching operations in powerhouse area. Beyond is upstream cofferdam, upon which several shop buildings have been located.

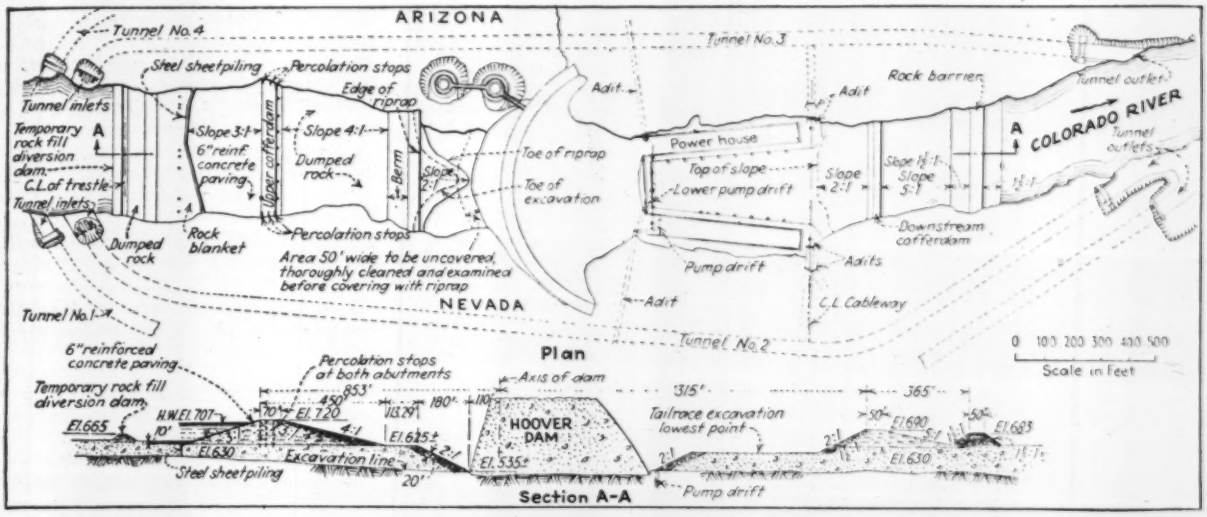


Fig. 2—Plan and section of dam site and cofferdams indicate extent of excavation and fill work necessary to prepare canyon bed for actual dam construction.

4 cu.yd. up to 21 of the largest-size units equipped with bodies having a capacity of 16 cu.yd.

Standard railroad equipment used to transport excavated material out of the canyon and bring in the earthfill for the cofferdams consisted of seven trains of seven cars each during peak operations. The 30-yd. side-dump cars were frequently loaded to 35 cu.yd. loose measurement. Double-track operations were used from the canyon terminal to the mouth of Hemenway Wash, where space is available to waste excavated material. Disposal was by side-dumping with gradual shifting of the tracks as more area was required.

Cofferdam construction

As soon as the river had been diverted from the canyon, the most important operation was to complete the upstream cofferdam to insure against any flood interrupting work in the dam site. The specifications estimated a volume of 568,000 cu.yd. of earthfill and 151,000 cu.yd. of loose rockfill to be placed in this structure, in addition to 3,500 cu.yd. of concrete in a 6-in. reinforced-concrete facing and a cutoff of steel sheetpiling along the upstream toe. The design of this 98-ft. dam, which does not differ essentially from the usual type of dam built of rolled earthfill placed in compacted layers, was outlined in *Engineering News-Record*, Dec. 15, 1932, p. 709. Its construction, aside from the speed at which the operations were carried on, was not unusual and need only be outlined briefly.

Two borrowpits ample for the requirements of the earthfill for both cofferdams were opened up at the mouth of Hemenway Wash near the established railroad line into the canyon. Two loops of track and a spur were extended into the pits, and one to three 3½-yd. electric shovels were used to load the railroad trains for the return trip after they disposed of the outbound load of excavated material. This feature of the railroad equipment moving loaded in both directions was an important factor in the speed and economy of operations during the period from November, 1932 to February, 1933.

Fig. 3—Activity was concentrated at the upstream cofferdam after its completion. The trucking road (lower left) leads up the back slope of the cofferdam to the loading docks at the railroad terminal. Extensive truck-repair shops and pits are shown in the foreground. The river is flowing into the Arizona diversion tunnels whose portals are at the right.

Excavation for the cofferdam was started behind a temporary diverting embankment (*ENR*, Nov. 24, 1932, p. 622), and about 100,000 cu.yd. was removed in October. The remaining 90,000 cu.yd. necessary to prepare the foundation was taken out in November, following diversion. This material was loose streambed deposit, including some bands of silt but principally sand and gravel. Removal of an 18- to 20-ft. depth over the cofferdam site produced a sound consolidated gravel surface suitable for the fill.

Difficulty arose in putting the steel sheetpiling down to bedrock along the Nevada side of the canyon, where a de-

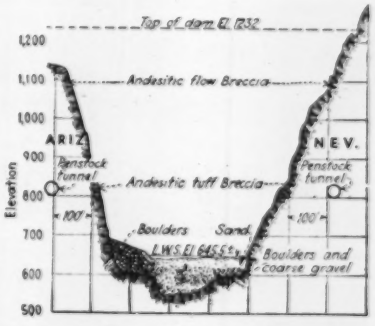


Fig. 4—Profile of canyon at the dam site, showing riverbed material and relative location of penstock and inner diversion tunnels. The slope of the canyon walls indicates the problems involved in abutment excavation.

posit of boulders finally made it necessary to excavate a trench all the way to bedrock, set the sheetpiling up and concrete them against the rock. This operation was carried out about a third of the way across the river, and then the decreasing number of boulders on the Arizona side permitted direct driving of the sheetpiling to bedrock or to consolidated tight material for most of the remaining distance.

Trainloads of earthfill, on arriving at the site, were dumped from a trestle into a stockpile and sprinkled as they were reloaded by shovel into trucks for placing on the dam. At times this material was handled so rapidly that it was impossible to moisten it properly before spreading on the dam, which necessitated some sprinkling as the material was rolled to compaction. Dumped from trucks, the material was spread in 12-in. layers by tractor bulldozers and compacted with sheep-foot rollers (Fig. 6).

The placing of 440,000 cu.yd. of earthfill in the upstream cofferdam during December required a concentration of equipment and crew which, although warranted because of the importance of this feature as the key to future operations, probably exceeded the economic rate. Average 24-hour operations placed about 14,500 yd., and on the maximum day 19,950 cu.yd. of earthfill was added to the dam.

While the fill was in progress the concrete facing was started and carried up the slope (Fig. 5) during January and February. Concrete was trucked to the crest and distributed to the point of placement by timber chutes and the 5-yd. dragline, which acted as a crane. Reinforcing steel amounted to 251,000 lb.

With the completion of the dam to its required height and the placing of the upstream paving, the structure was in shape to protect the site in accordance with the permanent diversion schedule. However, it lacked the final downstream blanket of dumped rock, which has been intentionally delayed because the downstream slope was used as the location of the truck road to remove excavated material from the riverbed. Further, the contractor was permitted to extend the original 70-ft. width of crest upstream to provide additional working area. This was done by first protecting the concrete paving with a 3-ft. thickness of sand and then adding a loose rockfill extending as far as the temporary diversion dam. The result provides a level area roughly 600 ft. wide by 400 ft. long, which is used for the various shop facilities.

Construction of the lower cofferdam was a later operation, and the earthfill was completed almost entirely during the month of February, when about 170,000 cu.yd. was placed. The majority of this material was handled in the same way as that for the upper dam, with the exception of the longer truck haul to the lower site, but in addition some of the material excavated from the streambed was of a quality suitable for this use and was included in the fill. The rockfill required for both dams and the material for the rock barrier below the lower cofferdam has been moved into place slowly as a secondary operation when equipment was available and averaged about 50,000 yd. per month. Some of the excavation for the lower cofferdam was disposed of directly by trucking downstream, and the remainder was removed upstream by the trains.

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Rock excavation

Excavation of rock from the canyon walls includes the following items according to the specifications: (1) loose-rock stripping of the walls, (2) excavation for the four intake towers, (3) excavation for the valve house and construction adit portals, (4) excavation of the power-house site and (5) actual excavation of the dam abutments. These scaling operations are all very similar in respect to the matter of attack and methods used. They have been described in previous articles and consist principally of jackhammer drilling and blasting beginning from the top of the canyon walls and working down along the required excavation line.

These crews work directly on the canyon walls with the aid of ropes and safety lines, although it is occasionally necessary to resort to boatswain chairs. Scaling operations are ordinarily only carried on during the day shift because of the hazards involved and the difficulty of providing proper light for night work. Although particularly spectacular and an important feature for the thousands of visitors on the work, this operation has not provided any outstanding new developments as to methods. The preliminary operation consists of prying loose all unsound rock, followed by drilling and shooting required to bring the excavation to the proper lines. Under maximum operations there were about 600 men engaged in this type of

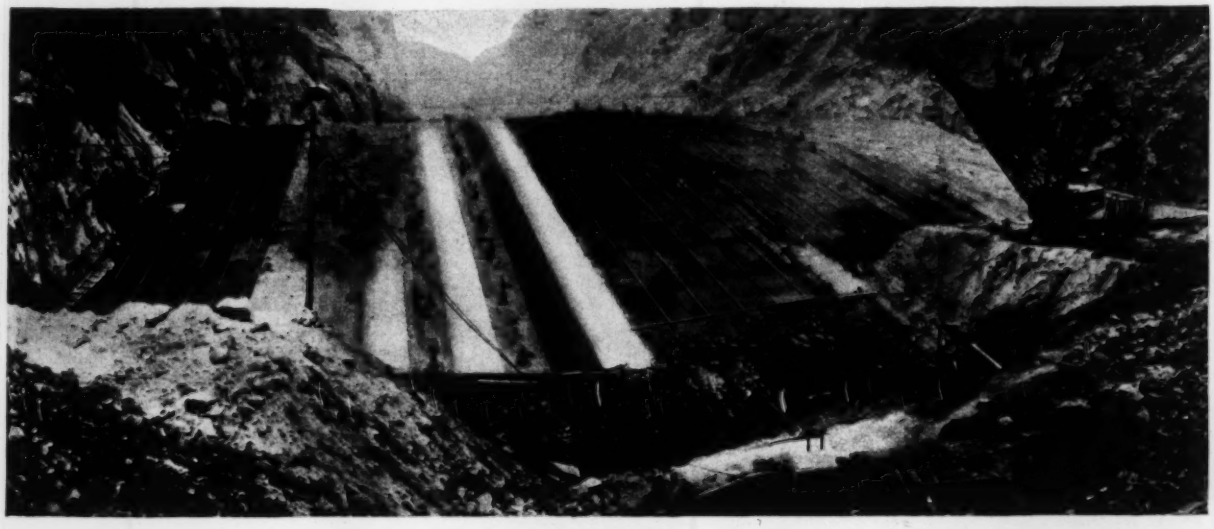

Fig. 5—Face of the upstream cofferdam during placing of the concrete paving in January. Note the driving of the steel sheetpile cutoff at the toe, the concrete pouring operations and the oil-drum salamanders on the fresh slab.

work, involving the use of about 100 drills.

The rate at which this scaling and other excavation work was carried on is indicated in the accompanying table. From a rate of about 55,000 cu.yd. during November and December, this work was speeded up to about 130,000 for the months of January and February. At the beginning of March 90 per cent of the wall-scaling operations had been completed, and the actual dam-site (abutments) excavation had been carried down to El. 675 (30 ft. above original low-water surface), representing about 50 per cent of the yardage to be moved. The excavations for the intake towers (Fig. 7) had also been practically completed except the final work of preparing the foundations for the towers. Power-plant site excavation had been delayed because of the necessity for keeping the portals clear while the penstock branch tunnels were being driven. Power-house site excavation will be speeded upon completion of this penstock branch tunneling, which is restricted on the Arizona side to driving from the canyon end because the inner tunnel is in use for diversion.

All of this canyon-wall excavation, when deposited in the riverbed, is handled with the same equipment and methods that were used in the cofferdam and riverbed excavation, with some secondary shooting.

Streambed excavation

Bedrock is required to be exposed from the downstream toe of the upper cofferdam through the dam site and the location of the power plant a distance of about 1,200 ft., in order to prove the rock formation in this area. In addition, more than 100,000 cu.yd. of rock will be excavated in the riverbed for the actual foundation of the dam. Specifications require that all this excavation be completed before any concrete is placed in the dam site. The table indicates that streambed material was excavated at the rate of 30,000 yd. in December, speeded to 94,000 cu.yd. in January and again doubled to 211,000 cu.yd. in February, with the rate of March operations remaining about the same.

This operation involves almost entirely simple truck-loading operations by electric shovels with the occasional shooting of boulders. The depth of streambed excavation averages about 40 ft. along the sides, with the additional 50 to 80 ft. of depth in the center of the river channel.

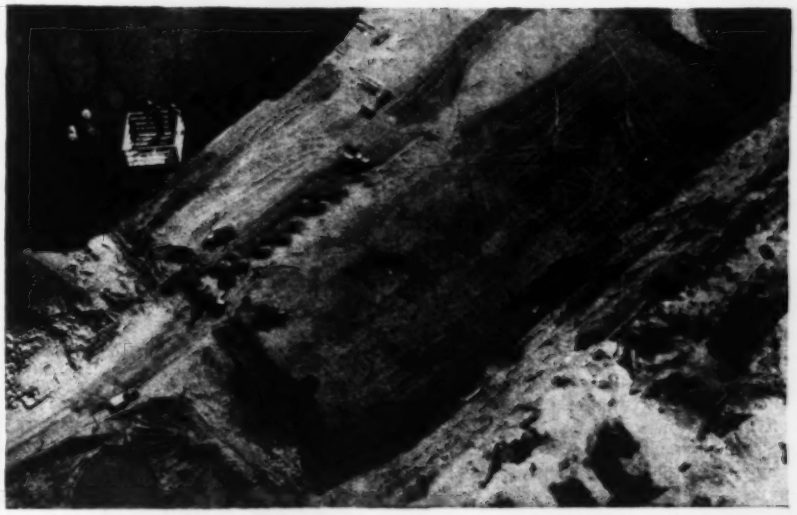

Fig. 6—Looking down on the cofferdam operations from a cableway skip. Note the dumped truckloads of earthfill equipment and sheer drop of the canyon walls.

General view of operations

A mere recitation of the various operations and the yardages moved, as stupendous as they are, does not provide much of an appreciation of the work. The concluding paragraphs indicate what these operations are like when witnessed on the job from an observation point on the upstream cofferdam.

Looking down the canyon, the wall-scaling crews can be seen on the precipitous rock through the film of dust from their drills. They are not far above the river level because canyon excavation has been completed above. Skips from the various cableways hover around these crews, supplying them with drilling supplies and transporting them to and from work. Loose rock is continually tumbling into the canyon from these operations, but the area is large enough so that it does not interfere with loading and trucking.

In the canyon area, which is about 450 ft. wide by 1,200 ft. long, the five big crawler-mounted electric shovels are loading trucks from the piles of loose rock that continually accumulate along the canyon walls or from the streambed gravel. There is no scarcity of trucks, and the shovels load with monotonous regularity, putting two dippersful into the small trucks to load them in 30 sec. and four or five dippersfull into the large-size 16-yd. body trucks. The continual stream of empty trucks is directed to the shovels by two "traffic cops" stationed on the road, so that the various shovels are supplied in accordance with their requirements. The loaded trucks, with the few exceptions required for miscellaneous filling at the downstream cofferdam, move up the back slope of the upstream cofferdam on a well-graded and sprinkled road wide enough for

Fig. 7—Shadows indicate the deep intake tower cuts. The one at the right is 300 ft. from top to bottom, and an idea of relative size can be obtained from the traveling cableway towers.

CONTRACTOR'S RECORD OF MATERIAL HANDLED AND MOVED DURING THE MONTHS OF MAXIMUM CANYON EXCAVATION

Type of Excavation	October Cu. Yd.	November Cu. Yd.	December Cu. Yd.	January Cu. Yd.	February Cu. Yd.	March Cu. Yd.
Riverbed (loose)						
Upstream cofferdam	108,000	87,000—completed				
Downstream cofferdam	.....	14,000	26,000	128,000	13,000—completed	
Dam and power house	.....	.....	29,000	94,000	211,000	387,000
Open-cut, diversion-tunnel portals	.....	.....	.....	.....	.....	10,500
Fill Material						
Upstream cofferdam (earth)	900	84,500	420,000	9,600—completed		
Downstream cofferdam (earth)	.....	.....	.....	2,700	172,000	24,000
*Upper, lower cofferdams and barrier (rock)	.....	.....	74,000	68,000	35,000	22,000
Canyon Rock						
Dam abutments	.....	.....	23,000	76,000	73,000	57,000
Wall scaling	15,000	27,000	21,000	14,000	3,000	2,500
Intake towers	.....	30,000	.....	40,000	46,000	3,000
Power-house site	.....	.....	.....	4,000	5,000	8,000
Total excavation for month	123,900	242,500	593,000	436,300	558,000	494,000

*This yardage of fill material is also included in the figure for "canyon rock."

three lanes of trucks. The grade is 12 per cent, and the crest is reached with only one sharp curve.

Up this grade the trucks roar in two lines, with the slower trucks on the outside lane and the faster trucks slowly passing them on the middle lane. The other lane is used by the empties returning downgrade, and their speed is a good measure of the rate at which this work is being carried on. Reaching the top the trucks move only a few hundred yards to the loading docks where the empty trains are waiting. Two loading docks are in use, and there is no delay. Spotters direct the trucks to the proper place for dumping, and as the loads slide into the car the empty truck is off again down into the pit. The procession is never-ending, and at maximum operations in February the loads moved out at the rate of about five per minute on the average throughout the 24 hours.

The extensive shops and repair pits on the top of the upper cofferdam are a busy place with a crew of two dozen mechanics always at work. This work

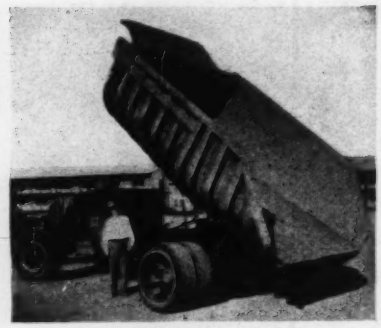

Fig. 8—Trucks with duralumin bodies haul 16 yd. of excavated material to railroad cars at the upstream cofferdam for disposed in Hemenway Wash. On record days 7,200 truckloads were handled.

involves not only the usual maintenance but the repair of the occasional unit that is towed in for more complete overhaul.

With the arrival of the lunch half-hour, operations stop and the equipment in the riverbed is moved, depending on the location and the size of the shot to be fired. For small shots the shovels are merely moved back a short distance, although all trucks and tractors remain up on the cofferdam crest. With the site cleared, the last carload of fire bosses leaves the site, and the crews cluster on the cofferdam eating lunch, while the familiar whistle sounds and the blast goes off, bringing down more material and filling the canyon with dust.

The dust hardly begins to clear when the tractors start roaring down the cofferdam road to get the loose rock off the road and the places for the shovels cleared up so that there will be no delay when loading operations begin again.

A visit to the operations after dark presents the same general picture although with a different touch. The illumination is sufficient so that the trucks operate without lights, although the shovels are equipped with lights to aid in loading. The cluster of lights is bright around the garage and repair shops, and there is a well-lighted area at the loading docks for the trains. There are no scaling crews on the walls, and the absence of the racket from their drills emphasizes the noise of the trucks.

Concrete Wall Replaces Levee at Helena, Ark.

Old 1914 levee surmounted by concrete wall, with sheetpile cutoff down to a 60-ft. depth, is substituted for enlarged 1928 section levee—Wall design and construction procedure

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IN DEVELOPING the levee protection of Helena, Ark., to the elevation called for in the Mississippi River flood-control project of 1928, now under construction, it was found that, due to the proximity of the river bank, permanent city improvements, railroads, industrial plants, and other improvements located along both river and land sides of the existing levee, it was not economically feasible to use the ordinary earth enlargement for a distance of about 3,161 lin.ft. Preliminary designs were prepared for various types of structures, and the final design adopted was a gravity concrete wall with steel sheetpile cutoff, or curtain wall, built on the top of the old levee, as indicated by Fig. 1.

Foundation conditions

The existing levee on top of which the concrete wall is founded was originally built in the '70s and was of small section, which was strengthened and raised from time to time until, in 1915, it was finally built to the Mississippi River Commission grade and section adopted in 1914. The earth for the levee was obtained from the adjacent Crowley Ridge bluffs which consist of loess that, due to its inherent characteristics and easy compaction, makes a good material for levee construction. The old levee has proved practically impervious throughout prolonged flood stages.

Analysis of foundation borings, taken along the land-side and river-side toes, as well as along the crown of the existing levee to depths of 40 to 50 ft. below ground surface, indicated a predomination of loess outwash from the Crowley Ridge bluffs, intermixed with the usual

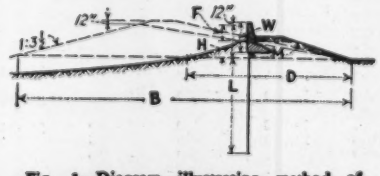


Fig. 1—Diagram illustrating method of computing depth of sheetpile cutoff for flood-protection wall built at Helena, Ark.

varying amounts of river deposit of sand, silt and loam. The borings through the existing levee and adjacent natural ground along each toe of the levee were made for two purposes: (*a*) to obtain soil samples for analyses from which to determine the classification of the existing levee section, as A, B or C provided for in the standard specifications; and (*b*) to ascertain the type of material through which the steel sheetpiles were to be driven, as a matter of information for prospective bidders. Analysis of these borings indicated a B cross-section, Fig. 1.

The plan and profile of the wall, as constructed, are shown by Fig. 2. The first problem was to determine the depth of the sheetpile cutoff to meet the requirement that it must be deep enough to provide seepage travel equivalent to that provided by the base of a B-section levee. Fig. 1 shows the B-section and illustrates the computation which follows. Let H=head above elevation of point at which seep line is tangent to and remains below land-side ground surface; B=base width of B-section earth levee for height of $H+1$ ft. (maximum water surface assumed 1 ft. below top of levee); D=existing earth base width at head H; L=length of piling required below elevation of base for head H; and W=height of concrete levee wall.

Then $B = (3\frac{1}{2}+6\frac{1}{2})\,H = 10\,H$. The length of seepage travel to be provided by piling $= 2L = B - D$ and $L = (B - D)/2$. Total length of piling required $= L + (H + 12$ in. $- W) + 15$ in. (projection of piling into concrete). Total length $= L + H + 2$ ft. 3 in. $- W$.

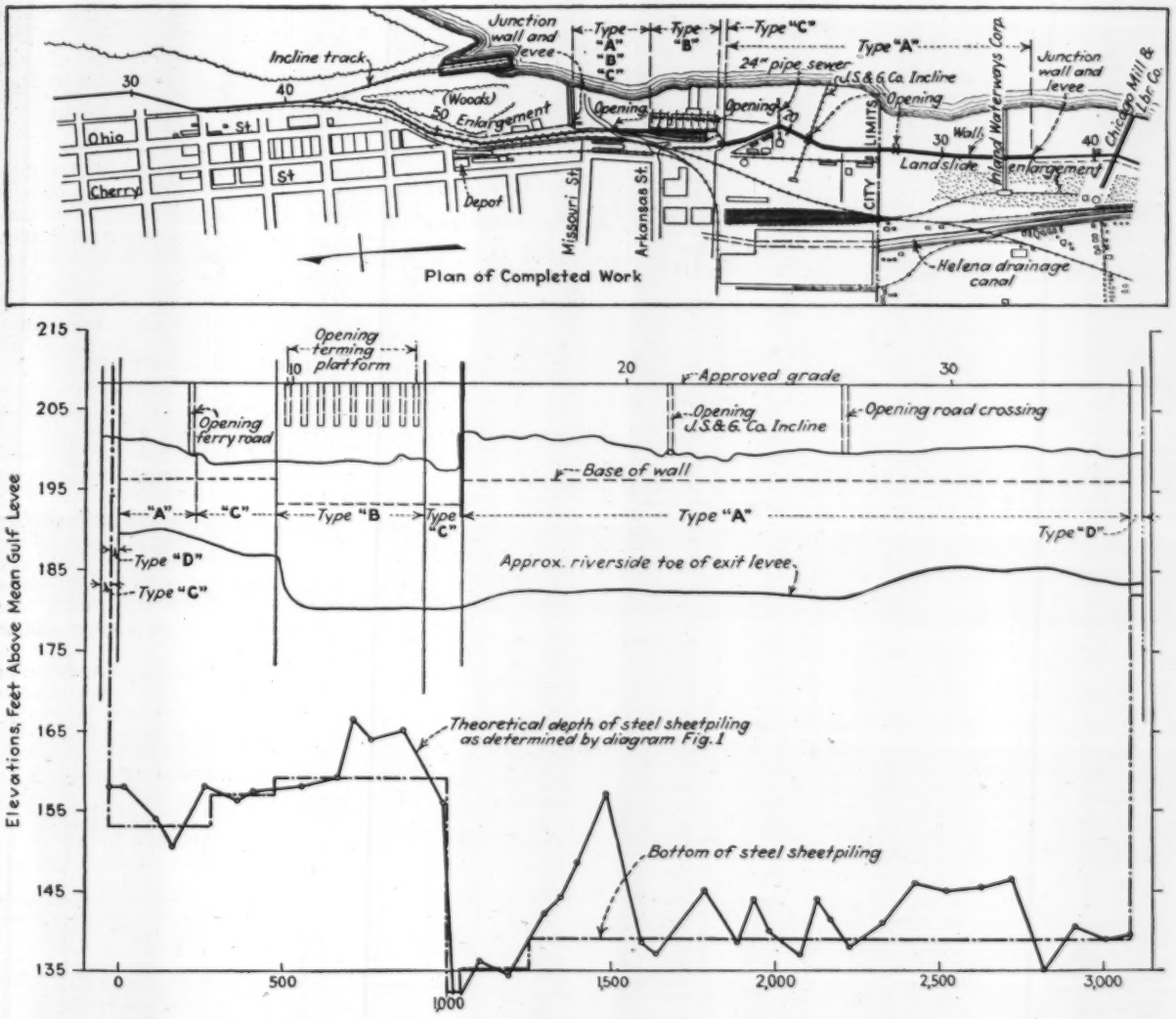


Fig. 2—Location plan and profile of flood-protection wall at Helena, Ark.

Also, let F = head at any point; $G = H - F$. M = shortest distance from pile to point on land-side, F distance below flow line. Then length must be such that $G + 2L + M =$ or $> 10F$.

From the above hypothesis of design the theoretical length of steel sheetpile cutoff was determined and is shown by Fig. 2, the variations in depths being due of course to variations of the existing levee section. The actual lengths of the steel sheetpiles as ordered and driven approached this line as closely as practicable.

Design of concrete wall

The concrete wall was designed as an ordinary gravity section, reinforced principally for temperature stresses and consisting of three distinct sections: A-section, 2,180 lin.ft. for a 12-ft. wall; B-section, 452 lin.ft. for a 15-ft. wall, modified to provide for stoplog openings for transfer of freight between warehouse and freight cars; and C-section, 383 lin.ft. for a 15-ft. wall. In addition, there were three special openings, aggregating 66 ft., and a D-section provided at the upper and lower ends of the wall, totaling 80 lin.ft., the latter being a continuation of the steel-pile curtain wall carried into the adjacent embankments as tie-ins.

The accompanying stress diagram, Fig. 3, shows the analysis for type A wall; types B and C were similarly designed. The nomenclature is evident from the diagram. The essential design requirements were: factor of safety against overturning (resisting moment divided by overturning moment) to be not less than 2; factor of safety against sliding (sheetpiling neglected) to be not less than 1. The sliding factor was computed as follows:

$$\text{F.S.} = \frac{C \times \text{net vertical component } R_v}{P_w \text{— passive pressure of backfill}}$$

in which C = coefficient of sliding, masonry on wet clay = 0.33; passive pressure of backfill determined from Coulomb's law, unit pressure $= W_e \tan^2 (45 \text{ deg.} + \frac{1}{2}\phi)$; ϕ = angle of repose of backfill = 18 deg. 26 min. for 3:1 slope.

Total passive pressure = $\frac{1}{2} \times$ unit pressure $\times H_e^2$. The maximum earth pressure on the base was computed by the formulas:

$$\text{Pressure at toe} = \frac{R_v}{l} + \frac{6R_v d}{l^2}$$

$$\text{Pressure at heel} = \frac{R_v}{l} - \frac{6R_v d}{l^2}$$

The maximum allowable earth pressure was 2,500 lb. per sq.ft.

The wall sections as designed are shown by Fig. 4.

Driving sheetpiling

The piles were unloaded from barges by a derrick boat and distributed along the wall by trucks and teams. The equipment for driving consisted of one crawler-mounted gasoline crane with a 114-ft. boom operating with a double-acting steam pile hammer, capacity

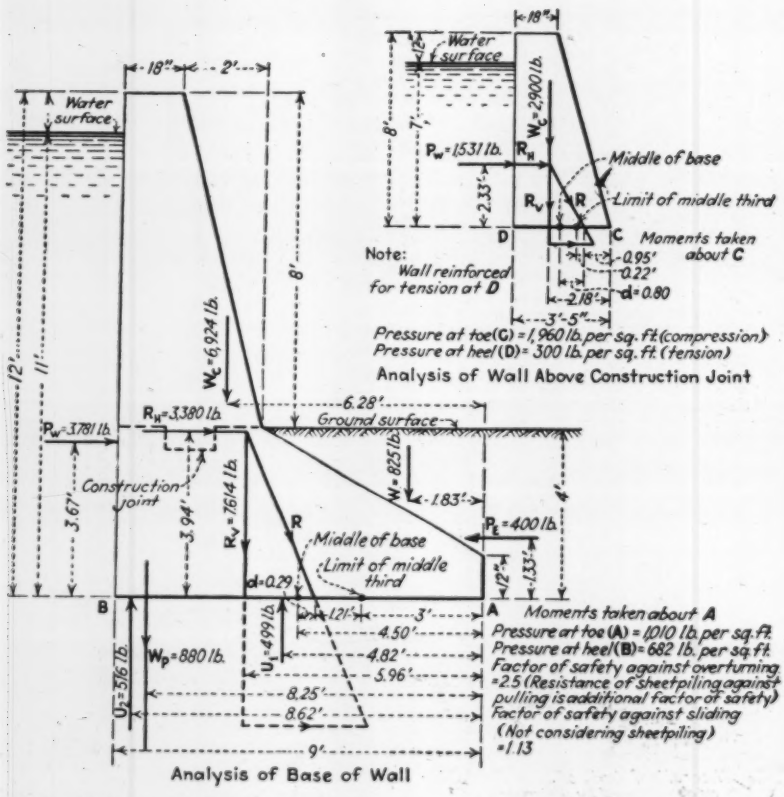

Water surface
Middle of base
Limit of middle third
Moments taken about C
Note: Wall reinforced for tension at D
Pressure at toe (C) = 1,960 lb. per sq. ft. (compression)
Pressure at heel (D) = 300 lb. per sq. ft. (tension)
Analysis of Wall Above Construction Joint
Ground surface
Construction joint
Moments taken about A
Pressure at toe (A) = 1,010 lb. per sq. ft.
Pressure at heel (B) = 682 lb. per sq. ft.
Factor of safety against overturning = 2.5 (Resistance of sheetpiling against pulling is additional factor of safety)
Factor of safety against sliding (Not considering sheetpiling) = 1.13
Analysis of Base of Wall

Fig. 3—Diagram illustrating computation of concrete wall for flood protection at Helena, Ark.

4,150 ft.-lb.; the other, a swinging derrick on skids with a 50-ft. boom, operating with a similar hammer, but having a capacity of 8,200 ft.-lb.

The established line of the wall of piling was kept straight by two heavy guide timbers rigidly blocked in a ditch. As each pile was threaded into the interlock of the preceding pile, it was plumbed in both directions. Laterally, each pile was braced to a heavy wood frame constructed of a 12x12-in. timber on each side of the wall, supported above the center of the piling, and by heavy poles both vertically and diagonally (Fig. 5). Whenever the piles tended to lean longitudinally in the wall, the bottom of the pile was pulled away from the preceding pile by a block and tackle, as it was dropped the last 12 in. before it reached the ground. When 12 to 14 piles had been set up and plumbed, they were driven in 8-ft. lifts to 40 per cent of the penetration. The whirley derrick then moved ahead, and the swinging derrick followed and drove the piling to the required penetration (Fig. 5). The small hammer operated at 180 blows per minute and drove two sheets at a time, at an average rate of 1 ft. in 1 min.; the other operated at 100 blows per minute and drove two sheets at an average rate of 1 ft. in 1½ min. The contractor employed two crews, each working eleven hours per day, who completed the job in a few days less than three months, or fifteen days ahead of schedule. The wages paid were average for this locality; the crews on the pile-driving averaged 27½c. per hour. The cost of driving the steel sheetpiling is given in Table I.

It is considered that the contractor used well-selected equipment except for the swinging derrick, which was mounted on skids. A traveling crane on caterpillar crawlers would have saved much time and cost. The above cost of 12.1c. per square foot is considered very reasonable, even though the contractor failed to make a profit.

Constructing concrete wall

The methods that the contractor employed to reduce the cost of the concrete work are worthy of note because each of these methods made better concrete, either in appearance or quality. The contractor saved money in making very sturdy forms and handling them very carefully. He was able to use one 50-ft. section of forms sixteen times, and on the entire job he obtained for all forms an average use of ten times, with no labor lost for rematching or reassembling. Each section was well spiked, and a plate was spiked on both ends of the studs to make each straight and rigid. The forms were carried from one location to another by traveling crane and were reassembled in exactly duplicate arrangement, thus avoiding the labor of drilling new holes for clamps and refitting. These sturdy forms produced a straight wall, free from bends, bulges and other similar defects.

The contractor saved much labor in rubbing the wall with carborundum bricks by the use of Masonite presdwood form liner, which produced a smooth uniform surface of wall equal in appearance to a carborundum-rubbed wall. This naturally formed smooth surface is considered better than a carborundum-rubbed surface, because it will withstand the elements and the absorption of moisture better. As the contractor used the forms as many as ten times, his saving over rubbing with carborundum was considerable.

The other economical feature of the

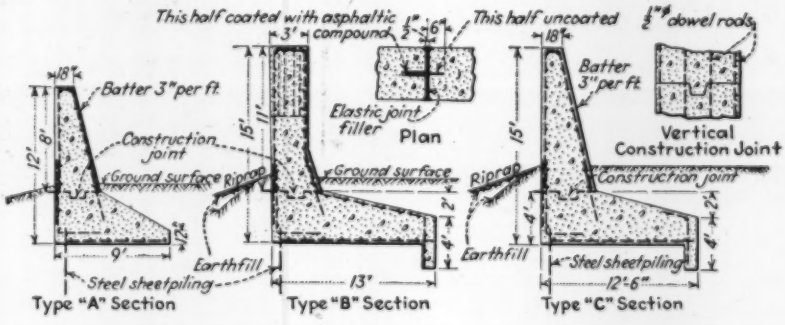


Fig. 4—Types of section employed for flood-protection wall at Helena, Ark.

TABLE I — COST OF DRIVING STEEL SHEET PILING FOR SEA WALL AT HELENA, ARK.

Item	Total Man-Hours	Cost per Sq. Ft. of Piling
Unloading: rental of derrick boat, truck and team hire and labor	2,298	$0.007
Driving: labor only	25,723	.044
Miscellaneous: coal, oxygen, acetylene, timber, etc	.....	.014
Mobilization: freight, hauling, labor, etc	2,776	.014
Plant rental	.....	.019
Excavation	4,456	.005
Losses: due to obstacles unforeseen, etc	138	.007
Field overhead and office	.....	.011
Totals	35,391	$0.121
Area of steel sheetpiling driven per man-hour		6.29 sq. ft.
Area of steel sheetpiling driven per pile-driver hour		63.1 sq. ft.

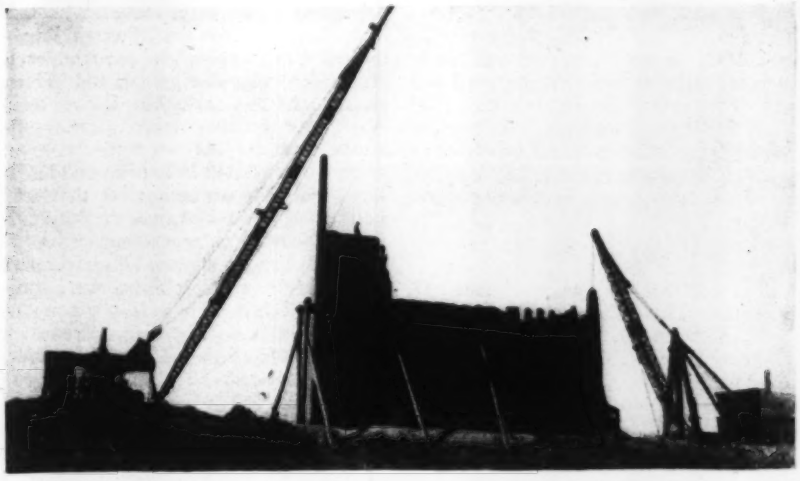


Fig. 5—Driving sheetpiles in two stages with two piledriving cranes, one driving to 40 ft. and the other to final depth, as shown by Fig. 6. Piles are braced to insure correct alignment in driving.

concrete was the use of local aggregates, which saved money regardless of the fact that the cement factor was raised because of their use from five bags per cubic yard, as specified, to an average of 5.59 bags per cubic yard for the entire job. The raising of the cement factor by the use of local aggregates increased the quality of the concrete. The local aggregates were good except for poor grading. The massiveness of the concrete would have permitted the use of 1½ in. aggregates; however, it never exceeded ¾ in. and contained large percentages of so-called pea gravel. The sand lacked the coarser sizes, having less than 2 per cent retained on the No. 16 sieve; and it lacked fine sizes, with less than 3 per cent passing the No. 50 sieve. Sometimes, when the coarse aggregate (gravel) contained much pea gravel, it was necessary to raise the cement factor to 6.5 bags per cubic yard in order to maintain the water-cement ratio and produce a workable mixture.

The slump of the concrete used in the footings was 2 in., and that in the walls was 3 in. Each batch was mixed more than 1½ min. in order to produce all the workability possible. It was placed in the forms and tamped into layers of not more than 4 in., producing the desired consistency. No special spading was required. Four laborers spread and worked by tamping 8 cu.yd. of concrete per hour, producing a smooth wall surface free from honeycomb, at a cost much less than spading. The proportions by weight averaged for the job 1:1.7:4.8.

Table II shows the relative cost of the various items entering into the cost of a cubic yard of concrete. This cost does not include reinforcing steel, which was not a large item, averaging only $0.704 per cubic yard. Table III gives the total cost of work to the contractor.

Although the method by which the contractor chose to do the excavation produced a small loss, the advantages gained in working space for piledriving and concreting more than compensated. Instead of excavating a trench just large enough to receive the concrete footing, the whole top of the old levee for about

Fig. 7—Section of completed wall and section being constructed, showing type and construction of wall forms.

Fig. 6—Top of line of sheetpiles is driven to grade, showing remarkable alignment secured by use of guide bracing (Fig. 5).

TABLE II — COST OF MATERIALS AND LABOR PER CUBIC YARD OF CONCRETE

Material	Units Used Per Cu. Yd.	Unit Cost	Cost Per Cu. Yd.
Forms: average labor $0.311 per hour, lumber $18.95 per MBF, presdwood 8c. per sq.ft.	11.33 sq.ft	$0.073	$0.827
Cement: new cost, including unloading, lost in bags, storage, etc.	5.59 bags	0.349	1.950
Coarse aggregates: dealer weights	1.23 tons	1.30	1.598
Fine aggregates	.459 tons	1.20	.550
Hauling cement from warehouse, aggregates from dealers' piles	.23 trans.-hr.	.91	.209
Labor: mixing and placing	4.27 man-hr.	.189	.805
Miscl.: expansion joint material, gas, etc.	.....	.....	.223
Plant rental	.....	.....	.068
Field office and overhead expense	.....	.....	.281
Total cost per cu.yd.			$6.511

TABLE III—COST OF CONSTRUCTING 3,161 FT. OF CONCRETE FLOOD-PROTECTION WALL ON MISSISSIPPI RIVER AT HELENA, ARK.

Item	Quantity	Unit Cost	Cost
Excavation	6,888 cu.yd.	$0.535	$3,688.04
Embankment	4,802 cu.yd.	0.392	1,882.56
Concrete	6,330.3 cu.yd.	6.511	41,226.47
Reinforcing	177,797 lb.	0.025	4,456.20
Structural Steel	27,537 lb.	0.027	751.20
Steel piling	161,711 sq.ft.	0.541	87,487.67
Riprap stone	6,089 sq.ft.	1.000	6,211.98
Timber stoplogs	12.995 MBF	45.638	593.06
Total contractors cost			$146,297.18

2,000 ft. was taken off by dragline to the approximate level of the bottom of the footing. This brought the actual amount of dirt handled to about three times the pay quantity. Dragline work was supplemented by hand excavation and tamping as well as by two teams for dressing the backfill. Because of the large amount of earth exposed and old sod disturbed by this method of excavation, the amount of sodding required was more than had been anticipated.

Although small in amount, embankment caused considerable work. This item consisted of bringing the river-side slope of the old levee under the municipal terminal to a 1 on 3½. This necessitated the handling of the material among piles, columns and cross-bracing. It was of course a tedious job, requiring much hand work. The cost to the contractor of handling the dirt was as much as the cost of hauling. The cost of embankment included the final dressing and sodding of the slopes.

Riprap paving was placed on the new fill under the federal barge line terminal and at both ends of the wall to reinforce the tie-in to the earth levee.

Timber stoplogs creosoted and cut to size for all openings were received and stored for future use in the cotton shed of the municipal terminal. Authorities of the levee board propose to construct a fireproof storage house for them, in order that they may be readily available.

Supervision

The contract was awarded to C. G. Kershaw Contracting Co., Birmingham, Ala., with D. M. Praytor, a director and executive of the company, in charge of the work.

Ralph E. Dippell, U.S. assistant engineer, was in immediate charge of field engineering and inspection under the area office. To his ability, initiative and rigid adherence to the plan and specifications is due the very high quality of workmanship. He was assisted by the necessary double-shift inspection force on layout, and other various items, such as forms, piledriving, concrete, etc.

Cold-Driven Rivets Show Superiority

Tests made in connection with construction of a 10,000,000-cu.ft. gasholder show that cold-driven rivets fill the holes better than do hot-driven rivets and develop a higher elastic limit, thereby permitting higher working stresses to be used

By F. G. Thorn, Jr.
Philadelphia Gas Works Co., Philadelphia, Pa.

THE USE of cold compression-driven rivets in place of heated rivets is being advocated by a number of steel-construction engineers and builders. One of the first large uses of cold-driven rivets was on a 10,000,000-cu.ft. water-type gasholder built for the Philadelphia Gas Works by the McClintic-Marshall Corp. In connection with this work parallel tests with cold and hot rivets were made, which revealed without question the superiority of cold driving. Better-filled holes resulted, higher yield points were the rule, and shearing strengths exceeded 75 per cent of the ultimate strength.

The heaviest work on the gasholder (bottom course of water tank and curb angle) required 1⅛-in. rivets in 2$\frac{3}{32}$-in.-thick plates. When joints occurred, the total thickness of the plate, butt strap and curb angle amounted to 5$\frac{13}{32}$ in. For this work 244 tons force was needed for driving the rivets, and 75 tons was required to draw the plates properly together. This necessitated the use of a No. 14-A compression riveter, capable of exerting 300 tons pressure. Such a riveter was used in the tests for the cold-driven rivets, while a No. 15 compression riveter with 75 tons capacity was used for the hot-driven rivets.

The rivets were made of standard boiler rivet steel, specified to be hot-made and self-annealed and to be "square-pointed." The heads of the rivets were button-shaped, with a diameter 1.5 times and a height 0.538 times the diameter of the rivet. The heads purposely were made no larger than actually necessary to develop the tensile strength of the shank of the rivet, and the relatively small height of the rivet head permitted the head end of the shank to be more readily expanded, thus filling the hole completely instead of deforming the plate around the hole. The field-formed heads were driven flat to the same diameter as the manufactured heads and to a height of one-third the diameter of the shank. Both the hot- and cold-driven rivets tested were taken from surplus stock at the gasholder.

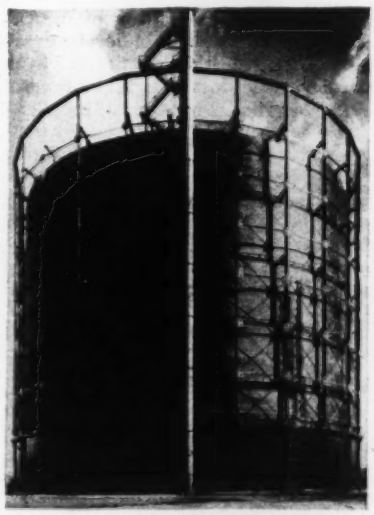
Specimens were made for both tension and shear test. The former were made up of four bars 4½ in. wide, 35 in. long and 5$\frac{3}{8}$ in. total thickness, in which seven 1$\frac{33}{64}$-in. holes were drilled. Cold-driven rivets 1⅛ in. in diameter were used in one specimen of this type and hot-driven rivets in another. In one hole in each specimen a split rivet was driven, affording means for comparing driven and undriven material. Thus two rivets were milled through longitudinally, and two halves, one from each rivet, were placed together and driven as though they constituted one solid rivet. After being driven, these split rivets were backed out and the rivet material was tested in tension and bending. The half of each rivet that was not driven was tested in a similar manner to provide comparative results.

For shearing tests two hot-driven and two cold-driven specimens were prepared. Each sample was made up of two 2x5-in. bars 22 in. long spliced with 1¼x5-in. bars 16 in. long on either side, the assembly being held together by one 1⅛-in. rivet each side of the joint.

Conclusions from the tests

The tests indicated first that rivets made of standard boiler rivet steel, hot-made and self-annealed, when driven cold are superior to hot-driven rivets due to the increase of strength developed by the cold working of the steel. Although the values for the hot-driven rivets were less than for the cold-driven rivets, they nevertheless met standard specifications.

The cold-driven rivets showed a decided advantage in shear over the hot-driven rivets; in double shear they developed an elastic limit 76.5 per cent greater, while the average ultimate strength was 6½ per cent greater. If, as is general, it is assumed that a working stress of one-half the yield point is proper, a much higher shear value for cold work is possible than for hot work.



Gasholder of 10,000,000-cu.ft. capacity at the Point Breeze station of the Philadelphia Gas Works Co., in which cold-driven rivets were exclusively used. It has a five-lift shell contained in a tank 265 ft. in diameter by 41 ft. deep. The tank bottom was completely arc-welded. The guide-frame columns are T-sections made by arc-welding two I-beams together. An elevator running from the ground to the top of the guide frame is a piece of equipment not ordinarily found in a water-sealed gasholder.

The actual test values were as follows: double-shear cold-driven rivets showed an ultimate shear strength of 44,795 lb. per sq. in. and an average elastic limit of 38,300 lb. per sq. in., or 85.5 per cent of the ultimate shearing strength; double-shear hot-driven rivets showed an average ultimate shearing strength of 42,045 lb. per sq. in. and an elastic limit of 21,700 lb. per sq. in., or 50.7 per cent of the ultimate shearing strength of the rivet.

The average shearing strength of the cold-driven rivets exceeded 75 per cent of the ultimate strength, but the hot-driven rivets were found to fall short of this figure.

The shearing tests provided justification for the use of small manufactured rivet heads, since the unbroken rivets underwent a considerable stretch without pulling off the head.

The tests of the material in the driven and undriven split rivets showed conclusively the changes in the characteristics produced by the driving, the most noticeable of which was the high elastic limit produced by cold driving.

The cold-driven rivets filled the holes better than the hot-driven rivets. In the case of some of the hot-driven rivets, the shank shrunk in cooling, leaving a void between the rivet and the plate, at the same time pulling the heads up tighter and giving a false hammer test.

Some tests made from field-driven rivets showed higher values than the rivets driven in the specimen bars, no doubt as a result of better driving conditions.

Letters to the Editor

Engineers in City Service

Sir—As you know, in these times of curtailment of government costs the engineering profession is being forced to carry a larger burden of lay-offs or reduction in salaries than any other part of the public service. May we ask you to give space to the following statement of the situation developing in the city of Newark, N. J.?

In May of this year practically an entire new board of commissioners was elected, and on June 6 there was introduced an ordinance to effect economies, the purpose of which is to lower drastically the compensation being paid to the engineers of the department of public affairs.

The 34 engineers in this group include the heads of the bureau of sewers, surveys, transit, lighting, docks, street cleaning and water, and in addition principal assistants and engineers in the several bureaus. The salaries of these men during the past year were subjected to deductions ranging up to 25 per cent in their base pay. This was a general reduction in salaries for the entire force of city employees.

According to the terms of the proposed ordinance, in lieu of this general cut of the engineers of the department of public affairs, it is proposed that cuts in the base pay of all engineers receiving $3,600 and over shall be approximately 40 to 50 per cent.

The engineers of the department of public affairs hope to have the terms of the ordinance now pending modified. We recognize the necessity for retrenchment of the costs of public service on account of the decrease in tax receipts, but we believe the burden should not be concentrated on the engineers. The work of the engineers requires specific training, which is not essential in other branches of the city service. In the city of Newark, particularly, the engineering department has never been affected by political changes in the governmental body. An *esprit de corps* has been developed in the engineering department which assures faithful and efficient service. This we believe the public will recognize and will agree with us that curtailment should be distributed throughout other branches of the city service and proportionately lightened on the engineering forces, and that any changes in base salaries should be temporary in effect.

MORRIS R. SHERRERD,
EDWARD S. RANKIN,
ELLSWORTH FRANCISCO,
Committee for Engineers Group, City of Newark.
Newark, N. J.,
June 12, 1933.

Public-Works Program Too Large

Sir—The group of articles contained in your Public-Works Construction issue of May 18 raises an important question: Have not the problems of taxation been given too little weight in the minds of those who are spending other people's money? How about the taxpayer as the forgotten man?

The experiment of business stimulation by public-works construction strikes me as proceeding on a far larger scale than now is warranted. Can we not accomplish all that is essential in the way of public works with an appropriation, say, of one billion dollars, in place of more than three times as much? I feel that it would be far better to start on that scale and to increase later, if necessary, than to go as rapidly as is proposed. Independent action has been criticized for having gone too fast, yet now the government proposes to go faster than ever before. We have a high opinion of many men in public life, engineers and others, and we feel that the government has its place, but at present it should not go too fast. In many instances the municipal credit will not warrant it, and the taxpayer is up against it.

There seems to be too little comprehension of the fact that heavy income taxes impair the extension of industry and the manufacturing security of industry. How can the world grow if it does not grow through income? And why do we seem to prefer that it should grow under government direction and management of income rather than through the instrumentalities that have produced the income?

Providence, R. I.,
June 5, 1933.
"INDUSTRIALIST."

Shallow-Deck Bridges

Sir—In a letter in the issue of May 11, 1933, Arthur N. Dowd suggests a design of a bridge which, he says would serve as an equivalent to the design described in my article in the April 6 issue. However, in his suggestion he neglects the fact that the bridge is subject to overflow by backwater, in which event his plate-girder design would act as a dam. The girders that he suggests would be at least 2 ft. deeper than the I-beams in the existing structure.

Also, in the case of future widening, the deck structure of plate-girder design would have to be scrapped, while the design used can easily be widened.

Allentown, Pa.,
May 22, 1933.
EARLE W. MECKLEY,
City Engineer.

Building Codes—A Correction

In the article on the proposed New York building code in the April 20 issue, p. 495, an error appears in the tabulation of the capacities of concrete-filled pipe used as piles. The error concerns the capacities allowed under the present, or 1915, code, which actually are somewhat greater than allowed under the proposed code. The tabulation in the article showed them to be considerably less. The correct tabulation is as follows:

	Pipe Diameters, Inches							
	10	12	14	15	16	18	20	22
Proposed code, tons	55	70	80	90	100	120	140	150
1915 code, tons	57.6	73.6	84.4	93.5	103.0	123.1	144.8	168.1

Vocational Guidance

In the notice of the publication of "Vocational Guidance in Engineering Lines," given in the book-review section of the May 25 issue of *Engineering News-Record*, the price of the book was given as $2. This was an error. The book should have been listed at $2.50. It is being published for the American Association of Engineers by the Mack Printing Co., Easton, Pa.

Engineering News-Record

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June 15, 1933 Volume 110 No. 24

Without Compulsion

THE VALUE of properly guided public opinion in regional planning work is well shown by the results obtained in the New York metropolitan district. The plan for that district is entirely voluntary and yet, as was pointed out at the annual meeting last week by George McAneny, chairman of the Regional Plan Association, nothing basically contrary to the plan has been done in the district since its adoption. During the years in which the plan has been in the process of development the public has been kept currently informed as to the steps proposed, with the result that those who have recognized the value of coordinating all new local development work with that of the whole region have been able to throw their support to such projects as closely approximate the ideal set up in the regional plan and have voiced opposition to major undertakings that depart from it. Thus, by the persuasive force of public opinion it has been possible to make notable progress upon a plan that could not as a practical matter have been imposed by compulsion upon the three states and the many scores of towns and cities within the area affected.

A Cornerstone Is Laid

WHEN the first bucket-load of concrete was dumped into the foundations of Boulder Dam several days ago the cornerstone was laid for the largest single piece of masonry ever erected by man, excepting possibly the Great Wall of China. Even the bulk of the famous Cheops Pyramid will be dwarfed by the mass of concrete to rise out of the depths of Black Canyon to hold back the waters of the turbulent Colorado. That first bucket of concrete marks the beginning of the final stage in the construction of Boulder Dam. Twenty-six months of intensely active preparation were necessary before a single yard of concrete could be placed in the dam proper. Chief among the items of preparation was the driving of the four huge diversion tunnels completed a year ahead of schedule. The next work was to throw up barriers to exclude the river from the dam site and to excavate the riverbed to the solid rock base that will carry the load of 3,500,000 cu.yd. of masonry in the dam. Building the cofferdams and excavating the dam site provided one of the greatest concentrations of earth-moving and excavation equipment ever witnessed in construction history. Batteries of shovels, trucks and railroad trains kept a constant stream of earth and rock moving out of the canyon bed, to and from Hemenway Wash and into the cofferdams. What seemed like hopeless confusion of equipment was in reality a most methodical and well-planned attack on one of the important phases of the building of the dam. Details of this attack are given elsewhere in this issue. With excavation practically completed, activities at Boulder Dam settle down to the routine of concrete placing. But this task itself is a colossal undertaking. Though construction history has already been made at Boulder Dam, there is more yet to come.

Specifications and Tests

AT ABOUT this time each year the importance to the engineering profession of materials specifications and standard methods of test is emphasized by the annual meeting of the American Society for Testing Materials. It is useful emphasis. The documents prepared by this body of producers and users are outstanding in their contributions to orderly technical and industrial development. Indeed, without them, uniformity of a product and predictability of its action in service would be difficult of attainment. And both of these are characteristics that a material must possess to be of maximum value to the engineer. Asphalt bridge plank, for example, comprises a product that has had no common specification or standard method of test during its first few years of existence. Each manufacturer has attempted to produce a satisfactory plank in his own laboratory, and each user has tried to guess at what kind of a plank he wanted. The result naturally has been a large variety of types of plank, some good, some poor and some failures. The experimental period has been costly to producer and user alike. Such a study of test methods and of specification requirements as L. H. Koenitzer outlines for asphalt bridge plank in this issue will do much to make realities out of the potentialities of this highly useful product. Any engineering material must pass through the same kind of investigation. The sooner a fairly acceptable specification and some standard methods of test are available, the sooner will the product assume its proper place. It is usually under the guidance of the A.S.T.M. that such work is carried out. Recognition of the importance of this organization to orderly progress in the technical world is an essential part of every engineer's education.

Save the Beaches

THE TAXABLES and income that the states possessing popular ocean beaches hazard by neglect are clearly indicated by the leading article in this issue. On 125 miles of ocean front the people of New Jersey have built property having a taxable value of $600,000,000. This investment brings in a revenue exceeded only by agriculture and manufacturing, most of it coming from people outside the state. Without the beaches this investment and this income would not exist. Yet, except in almost negligible measure, New Jersey as a commonwealth fails to interest itself in the protection of so valuable an asset. In this New Jersey is not unique, as no other ocean-bordering state does more.

This situation comes from the traditional assumption that the beaches are solely a private property interest. In fact, however, the beaches are the concern of the community and of the state to be maintained and preserved for all citizens. If left to private powers alone, they cannot be preserved.

Beach-protection engineering is not an assemblage of individual structures built where, when and how private

owners may choose, and not built at all by many. It calls for a system of structures embracing long sections of beach. Beach erosion and littoral sand movement on the ocean coast, as the article in this issue clearly demonstrates, are not local occurrences but coast-wide phenomena, and their control must be planned on a corresponding scale. Construction can go ahead in detail, and the points of greatest hazard can be protected first, but as parts of a planned system and not as isolated individual works. This is a task that falls definitely within the authority and public-works responsiblity of the state.

Engineering skill is capable of the task of preserving the beaches. With studies such as are being carried on by the New Jersey state board of commerce and navigation and the federal board on coast erosion and beach protection the fundamental principles and hypotheses are gradually being developed that will make coast engineering constantly more positive and definite. Enough success has been had with the few broadly planned works already completed to demonstrate that the beaches can be preserved and built up to increased values. But the task cannot be left to the puny efforts of individual beach-front property owners. State financing and engineering are the only adequate means.

Cooperation for Recovery Begins

CONGRESS passed the Recovery Bill early this week with only minor changes, as had been expected. With this historic closing act of an historic session, it set in motion the most fundamental and far-reaching step in the program of rehabilitation and recovery. Congress has made a beginning in a campaign which all citizens and all governmental units must now join in bringing to successful execution.

The work of Congress in connection with this remarkable act should not be under-rated. The legislators were by no means mere rubber-stamp endorsers of a prescribed plan, but on the contrary they entered into thorough study of the act. The very confusion and conflict of the closing sessions of Congress was in itself evidence that the inevitable differences of view concerning the measure were being brought out for consideration and that the final outcome expressed the undoubted conviction of Congress and of the people at large. The passage of the bill was a shining example of nationwide cooperation.

When the mind turns back to the hopes that were entertained as to the constructive results of last year's relief and construction act, it is easy to see that any measure of stimulated recovery may fail if left to itself or administered in lax manner. So the effectiveness of the Recovery Act now will depend on the energy and purpose applied to its execution. The passage of the bill is therefore at best only a beginning. Let us keep in mind at all times the purpose of the bill—to return idle workers to employment and thereby to revive industry and business. Its purpose is not to furnish temporary occupation, not to provide charity or made work in a new way, but to restore the normal activities of all fields of production and trade and to return each worker to his own occupation. The rapidity and thoroughness with which employment is provided and the degree to which the normal methods of construction and business are brought back into action are thus of vital importance.

Briefly, the bill itself does two things; first, it authorizes and directs the construction of public works of federal, state and local governments, up to an amount of $3,300,000,000; second, it authorizes and directs industrial concerns to cooperate to the end of increasing employment and wage income, and for this purpose it provides powerful means of enforcement, subject only to the limitation that monopolistic practices must be avoided. Both sections of the bill plainly are directed toward regenerating projects and rescuing business from ruinous competition, bad price conditions and unbusinesslike ways.

That the undertaking is of tremendous magnitude may be judged from these facts alone. But in addition the task is measured by the huge figure of more than twelve million unemployed. Fullest possible application of the construction section cannot be counted on to restore work to more than three or four million of these; but the workers employed through construction will in turn have to bring back the remainder—the merchants, clerks, and in short the whole staff of the service and supply vocations. This is not the process of a day or a month, however; it will require many months at best. But it will proceed most rapidly if full cooperation is brought to bear in making the act effective—cooperation in every state, city and household. The same spirit of cooperation displayed by the Congress and the President must now animate our other governments, our people and our industries.

A profound change has come over the attitude of local governments and the people since a year ago. Where last August general apathy greeted the enactment of the Relief and Construction Act, the spirit now prevailing is one of intense willingness to seek out works of construction that are ready to be done, in every part of the country, and to have them ready to contribute to the effectiveness of the Recovery Act. This spirit, wisely and effectively directed, will make the success of the latter act certain.

Industry also has set to work intensely in the three weeks since the act was first proposed, studying how it could most effectively adjust itself to the new requirements. Practice codes have been drafted, organizations have been worked out and in part established, and many industries are ready to go. Perhaps industry is more in danger of mistaking the purpose of the enactment than are the local governments and citizens, for some industrial groups seem to have proceeded as though the chief or sole purpose of the industry-control section was to restore prices and decrease competition. Such misconceptions are only natural in view of the unprecedented character of the act and the complete about-face it makes in all our industrial thinking. In the operation of the act, governmental supervision should not find it difficult to reorient purposes of these groups to the primary objective of enabling industry to increase employment and wages and reduce working hours. Increased volume of business and increased profits can come only as a secondary result, based on improvement of the workers' status and the restoration of normal purchasing.

An organizing job of vast size lies just ahead in preparing to administer the act. A remarkable amount of preliminary work has already been done, but much more remains, both in respect to public-works construction and in respect to the revival and reorganization of industry. Since the whole effectiveness of the act depends upon the rapidity with which it is made to function, the cooperation of all in developing the required organization and putting the act to work is essential.

NEWS OF THE WEEK

Organization Set Up To Expedite Action Under Recovery Act

IN PREPARATION for putting the Recovery Act into effective operation immediately after its passage, active planning and paper organization work was begun nearly a month ago. This applies to both the industry control section, Title I, and the public works section, Title II. Hugh S. Johnson was called upon by President Roosevelt to undertake this planning and organizing. He gathered about him a group of specially qualified men, partly detailed by government departments and partly volunteers from private life.

Public works and industry control

From the beginning this staff was divided into two separate groups, one handling industry control, the other public works. The former established contact with major industries, inviting them to draft tentative practice codes and outline the most effective form or organization for their industries. It also discussed with many of these groups the drafts and organization schemes thus prepared. One primary aim in this work was to guide the industries towards developing codes and structures adapted to the specific purpose of the Recovery Act pending before Congress, namely, "to reduce and relieve unemployment." Another purpose was to make the operation of federal supervision workable by inducing industries to organize in few large units rather than many small ones. Much progress was made in these directions, and it is estimated by those in a position to appraise the situation accurately that the preliminary work has advanced the date when the control section will be effective by many weeks.

Public works divisions

Even more extensive preparatory work was done by the public works groups. The staff assembled by Gen. Johnson was arranged in two divisions, one dealing with highways, rivers and harbors, and general federal works, while the other dealt with city and state public works other than highways, and with housing and private construction for public use.

Spalding and Waite chosen

Both divisions were headed by Col. George R. Spalding, division engineer of the Upper Mississippi River Division of the War Department and member of the Mississippi River Commission. Col. Spalding took personal charge of the federal works, and delegated state and municipal works to Henry M. Waite, general manager of the Cincinnati Union Terminals Co., and former city manager of Dayton and Cincinnati successively.

To mobilize a full "immediate" program of federal construction, Col. Spalding drew on the assistance of the Federal Employment Stabilization Board, the Director of the Budget, and all the government departments. With their aid a carefully selected list of federal projects ready to go ahead immediately was compiled, totaling slightly less than $400,000,000. The items on this list were selected on two considerations: that the project could be started immediately, and that it was sound as a public investment, i.e., that it would not increase the expenses of government. Only a small number of public buildings of the post office type were included. A definite acceleration of the basic survey work of the government was thought essential, in view of the fundamental character of survey data to all construction and fixed investment. Several million dollars were allocated to geodetic work, and to topographic surveys.

State and municipal work

By far the largest responsibilities, however, were known to be attached to state and city public works. It was necessary to compile as rapidly as possible a complete list of works ready for construction and select them according to priority. A call was made on governors, mayors and other local executives to develop lists of ready work, on the basis of stated priorities (ability to start work at once; providing work at points of greatest unemployment; spreading work over a wide territory; reaching as many crafts as possible).

Simultaneously, complete skeleton organization plans for the operating staff of this division were worked out. The controlling objective set here was to decentralize authority and function in fullest measure, and thereby to avoid all danger of bottle-necking the operation of the act in reviewing offices. The characteristic feature of the organization is a group of 47 district administrators, generally one to each state. The office of each district administrator will be a completely functioning organization, with its own financial, legal and technical advisers, accountants, inspectors and the like. To relieve these offices of the burden of disbursing responsibilities the plan was adopted of appointing local disbursing offices of the Corps of Engineers, U. S. Army, as disbursing agencies, making payments on the vouchers of the district administrators.

Regional groups

Auxiliary to this direct-line organization are ten regional or divisional groups comprising a field engineer, and a division examiner in the Washington office serving the purpose of promoting and expediting. These are auxiliary to the direct operating system of district administrators and are not interposed between the district offices and the chief administrator. A further auxiliary is a housing division, reporting direct to the administrator. Another is a division of private projects. Among its responsibilities will be dealing with the several hundred remaining projects in the hands of the Reconstruction Finance Corporation, whose responsibilities for construction have been abolished by the Recovery Act.

This public works organization was perfected fully a week before final passage of the act, and could have been put into operation on 24 hours notice. In the meantime detailed instructions with respect to interpretation and application of the act had been prepared for the guidance of district administrators. Organization schemes for the district offices have been outlined and the principal forms and schedules have been drafted.

District responsibility

Under the system thus set up and now going into operation, the district administrators are direct representatives of the chief administrator and exercise his full power. In other words, subject to the veto by the administrator alone, the district administrators make all the actual decisions and carry out all contracting and supervision as well as approval of the technical, financial and legal details of projects in their territory. They control all advances. The system is keyed to speed and expedition and to quick adjustment of legal, financial and technical complications.

Under the remarkable powers conferred on the President and the chief administrator by the Recovery Act, public works may be financed wholly or in part, or may

Johnson, Spalding and Waite To Administer Recovery Act

Hugh S. Johnson, former member of the War Industries Board during the World War was selected some time ago by President Roosevelt to head up the administration of the Industrial Recovery Act. He has chosen as his principal aids Colonel George R. Spalding, division engineer of the Upper Mississippi River Division and a member of the Mississippi River Commission, and Henry M. Waite, the engineer who recently carried through to successful completion the railway terminal reconstruction at Cincinnati, Ohio. The official appointment of these men is expected as soon as the act is approved.

Hugh S. Johnson

be both financed and built by the administrator. Grants of aid not required to be repaid may be made to public authorities in an amount not exceeding 30 per cent of the cost of labor and materials. The policy decided upon by the President under the terms of the act is to make no differentiation between public bodies as to the price of their bonds accepted by the administrator, all bonds being accepted at par. A uniform interest rate will be fixed, assumed for the present at 3½ per cent.

Under the same provisions the grant may be used to reduce the bond obligations of the public authority or may, in part, be used to cover interest and amortization during the first years after the completion of the project.

The administrator also may build a project under agreement with the local government to purchase the project on installment payments after completion, or he may lease the project to the local government.

Call for Bids on Disposal Plant for Columbus, Ohio, Authorized

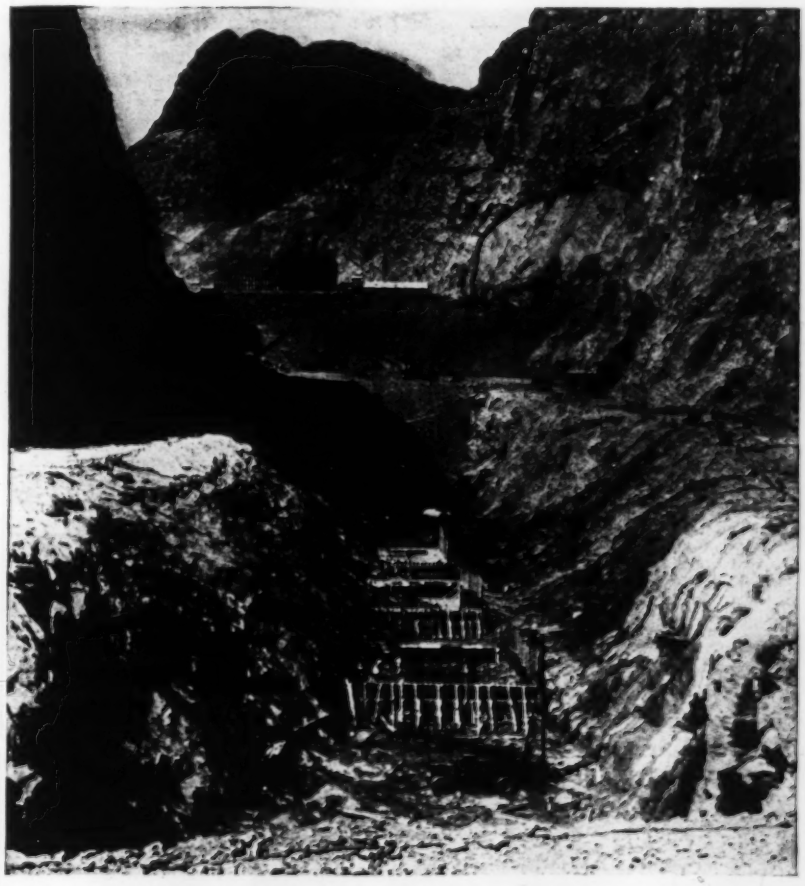
The city council of Columbus, Ohio, on June 5, authorized the city service director to advertise for bids and to enter into a contract with a private corporation for treatment of the city's sewage in a plant to be financed, built and operated by the successful bidder.

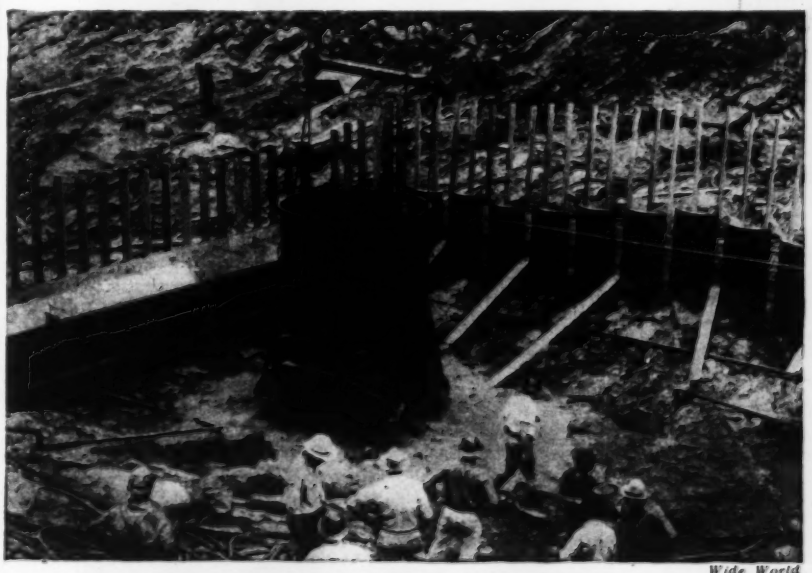
As has been noted in previous issues, the subject of sewage disposal at Columbus has been actively considered for some time because of the State Health Department's order to the city to provide adequate treatment for its sewage in order to end pollution of the Scioto River. Construction of a municipally owned and operated plant and construction by private parties both have been considered, but the latter was adopted on account of the difficulty of financing municipal construction work at the present time.

$2,500,000 Housing Project Proposed for Dayton, Ohio

The unit committee of the Council of Social Agencies of Dayton, Ohio, has applied to the Federal Emergency Relief Administrator for a loan of $2,500,000 for a housing and homesteading project in that city. The project calls for the establishment within the coming year of 50 homestead units of 35 to 40 families each, similar to the unit already established and known as the First Homestead Unit of Dayton, Ohio, Inc. All the units are to be located in a circle around Dayton on tracts of land which can be secured at prices low enough to make the plan self-liquidating, a maximum limit of 15 miles from the center of the city has been set so that the people living on the homesteads can work in the city.

The first unit is being financed locally by means of an issue of "independent bonds" bearing 4½ per cent interest and maturing in 15 years. The families for this development already are on the ground. Actual building of houses is to be done by the homesteaders themselves under the direction of the unit committee's engineers and architects.





Wide World

FIRST CONCRETE PLACED IN BOULDER DAM

Last week the first concrete was placed in Boulder Dam in the Black Canyon of the Colorado River after 27 months of intensive preparatory work. Contract for the work was let to Six Companies, Inc., on March 11, 1931.

The upper picture shows the bottom of the river channel as exposed after the completion of the excavation work described elsewhere in this issue. Forms for the first blocks of concrete were being erected at the time the picture was taken. In the lower picture, the first bucket of concrete is being placed in the bottom of the canyon, many feet below river level. The dam will contain 3,500,000 cu.yd. concrete.

Federal Trade Board Reports on the Cement Industry

The Federal Trade Commission transmitted to the Senate on June 9 the report of its investigation of the cement industry carried on under a Senate Resolution adopted Feb. 16, 1931. The commission was instructed to report on the price and distribution activities of cement trade associations and whether such activities constitute a violation of anti-trust laws of the United States or constitute unfair trade practices.

The report contains a summary of price determination and distribution practices, but does not express the commission's conclusions as to the legality of activities as carried on by the cement industry. The commission did, however, express the conclusion that there was no monoply in the cement industry.

The recommendation is made in the report that the cement companies quote prices f.o.b. mills, based on costs at each mill, which would eliminate the uniform price system now in effect and would establish price competition. Further investigation is being made as to the legality of trade practices. The report will be abstracted in more detail in a subsequent issue.

Detroit Council Plans Action on Large Construction Projects

In anticipation of the passage of the Industrial Recovery Act, the council of the City of Detroit, on June 6, adopted a resolution calling upon the department heads to prepare by June 15 a report on projects which might be initiated under that act, also to indicate which of the projects would be of a self-liquidating character. The council's action was taken with special reference to two major construction projects put forward by the Mayor's Committee on Projects for Employment.

The two major projects put forward by the mayor's committee were (1) betterment of public transit facilities and (2) betterment of sewage disposal systems. Under transit facilities the committee calls for the construction of 20.6 miles of 2-track rapid transit subways estimated to cost, at 1933 prices, $87,853,000. Annual charges would be met by an increase in fare of two cents commencing in 1937 when the new mileage would go into operation.

The sewage treatment plant called for is estimated to cost $12,500,000 and would complete the work already started through the construction of $5,000,000 worth of intercepting sewers. The sanitary sewer system would be financed by charges against the users.

Other projects recommended by the city were the completion of the Springwells pumping plant at a cost of $1,600,000, four incinerators at a cost of $1,800,000, and the construction of 22 grade separations on five principal thoroughfares in the city and at other important points. The cost of this work is estimated at $50,770,000 which would be divided between the state, the city and the six railroads concerned. The city's share would be about $7,800,000.

Extensive riverfront improvements also were outlined in the report.

Bids Called for Water Tunnel

The Twin Lakes Canal & Irrigation Co., Ordway, Colo., has called for bids on June 22 for the construction of 3.6 miles of tunnel together with canals to divert water from the Roaring Fork under Independence Pass to the headwaters of the Arkansas River to be stored in the Twin Lakes reservoir. The work is being financed by the Reconstruction Finance Corp.

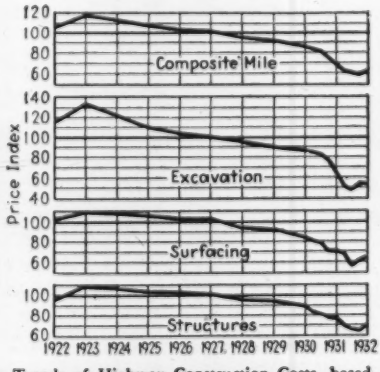
Cement for Missouri Highways Bought at December Prices

After having been held up for nearly six months, contract for the purchase of about $2,000,000 worth of portland cement for the highways of Missouri was put through early in June. Final letting of the contract has been held up by an investigation to determine whether the Missouri anti-trust law had been violated by the cement companies when bids were called last December.

During the period that the investigation has been in progress, cement prices have risen and the highway commission recently advised state officials that cancellation of the contract would result in a considerable increase in the cost of cement to the state. In the meantime, the cement companies have accommodated the state by making needed shipments at the December prices.

Eleven Year Trend Shown in Highway Construction Costs

Indexes of highway construction costs for the years 1922 to 1932, inclusive, have been compiled by the U. S. Bureau of Public Roads and published as a chart in its May issue of *Public Roads*. The chart, showing the cost indexes of a composite mile, excavation (grading), structures and surfacing is reproduced herewith. The



Trends of Highway Construction Costs, based on 1925-1929 Federal aid costs.

general or composite mile curve is an index in which the weights assigned to the prices of the various items used are the amounts of these items which would have been used in building one mile of road, if that mile had contained an average amount of grading and footage of structures and had the surface been composed of gravel, macadam, concrete, etc., in amounts proportioned according to the average usage of these various surfaces. The base period used in determining both weights and average base prices in compiling the indexes is the period 1925 to 1929, inclusive.

E. P. Goodrich to Have Charge of Sanitation Work at New York

As part of the reorganization of the sanitary work of the city of New York, George McAneny, recently appointed head of the sanitation department, has named Ernest P. Goodrich, consulting engineer, New York, to be head of the engineering division of the department. Mr. Goodrich will be charged with the completion of Ward's Island sewage disposal plant and the construction of new incinerators to permit the city to comply with the Supreme Court's order to cease dumping garbage at sea.

Mr. Goodrich is a native of Brooklyn and received degrees in civil engineering from the University of Michigan in 1898 and 1901. For a time after graduation he was in the civil engineering department of the Navy. Subsequently he became chief engineer of the Bush Terminal, Brooklyn, serving as such from 1903 to 1907. From 1910 to 1917 he was consulting engineer to the Borough of Manhattan and since that time has been largely in consulting practice both in this country and abroad.

One Trestle for Three Railways Proposed for Spillway Crossing

The federal government, in a motion to dismiss the damage suit recently brought to establish the cost of erecting separate structures for the Louisiana & Arkansas, the Yazoo & Mississippi Valley, and the Illinois Central railroads over the Bonnet Carré spillway channel above New Orleans, has announced that it will enter into negotiations with the railroad companies in order to determine whether it will not be possible to build one trestle to carry all three railroad lines across the flood channel. The weir at the head of the spillway was completed some time ago and the side levees are completed except for the openings left for the railroad tracks and the highways which now cross the spillway channel at bed level. It is not expected that the spillway will have to be used this year.

Several weeks ago the commission set up to determine the amount of damages to be awarded the railroads made a finding of $1,366,470 for the Yazoo & Mississippi Valley R. R. and of $1,378,161 for the Louisiana & Arkansas. The two railroads had sought damages in excess of $2,500,000 as the estimated cost of building trestles across the 9,000-ft. spillway channel. An award for the Illinois Central had not been made at the time the motion to dismiss the damage suit was entered.

Opposition to the federal government's motion to dismiss the proceedings was filed in the federal district court by the Louisiana & Arkansas Railroad within a short time after the filing of the government's motion. The railroad claims that by virtue of the original petition for the establishment of damages and by other acts, the federal government has acquired flowage and levee rights across its property and that its trains now operate through the spillway by sufferance of the government. The railroad cites further that the spillway has been virtually completed, that the material to close guide levee gaps has been placed at hand and that the structure will be brought into use whenever necessary to prevent the Mississippi River at New Orleans from passing the 20 ft. stage.

Consolidation of Railroad Lines in Southern New Jersey Approved

The Interstate Commerce Commission, on June 10, authorized the Pennsylvania R.R. and the Reading R.R. to consolidate their lines serving the southern part of the state of New Jersey. The principal lines affected are the two closely parallel lines between Philadelphia and Atlantic City.

Under the proposed consolidation, the Camden terminals will be considerably changed, only one line will be operated between Camden and Atlantic City, a new union station will be built in Atlantic City and all duplication in branch line services will be eliminated. The operation of the lines kept in service will be carried out through a subsidiary.

Loan Approved for a Bridge Across Apalachicola Bay

The Reconstruction Finance Corp., on June 8, agreed to purchase $850,000 of 5 per cent bonds of Franklin County. Fla., to aid in the construction of a toll bridge for highway traffic across Apalachicola Bay. Application for the loan was made jointly by the Florida State Roads Department and Franklin County. It was one of eleven loans totalling $2,811,500 approved by the corporation on June 8.

Among other loans authorized were eight self-liquidating ones as follows: $133,300 to the city of Bedford, Ohio, for the construction of a sewage disposal plant ordered to be built by the Ohio State Board of Health; $175,000 to the Southeastern Merchandise Market, Inc., for a central market for display of textiles and associated products at Charlotte, N. C.; $60,000 to the city of Angola, Ind., for the construction of a new sewage disposal plant; $65,000 to the town of Herington, Kans., for a like purpose; $25,000 to the town of Berryville, Ark., for a new source of water supply; $3,000 to the Crowell Independent School District, Foard County, Texas, for a high school gymnasium, and $500 to the Yoakum Independent School District, Texas, for machinery for a canning factory. The latter loan is the smallest yet made by the R.F.C. for a self-liquidating project.

The proposed sewage treatment plant at Herington, Kansas, is notable in that the effluent is to be sold to the Rock Island Railway for boilerwashing purposes, thus permitting the railway company to abandon its present pumping station and softening equipment. The city will construct a complete activated-sludge sewage treatment works consisting of a raw sewage pumping station equipped with mechanically cleaned screens and three 350 gal. per minute sewage pumps, a preliminary settling tank with mechanical equipment for sludge collecting and grease removal, aeration tanks and a final settling tank.

The directors of the R.F.C. on June 8 also authorized an additional commitment of $500,000 to the Unified Rehabilitation Corporation of Long Beach, Calif., to be used for loans for the reconstruction of buildings destroyed during the earthquake and loans of $400,000 to the Ellsworth (Maine) Rehabilitation Corporation and $600,000 to the Auburn (Maine) Rehabilitation Corporation, two non-profit organizations, for the reconstruction of buildings destroyed by fire which swept both communities this spring.

SPHERICAL WATER TANK SUPPORTED BY TUBULAR COLUMN

On the campus of Emory University, Atlanta, Ga., there has recently been erected a 100,000-gal. water tank consisting of a 30-ft. steel-plate sphere supported on a 100-ft. high tubular column. The column is 15 ft. in diameter at its base, tapering to 8 ft. at the top. An 8-in. riser pipe inside the column serves the tank. In erecting the tank the scaffolding was built inside the tubular column as the pieces were riveted together.

The sphere itself has been painted with aluminum paint, the support is painted dark green. The tank was designed and erected by R. D. Cole Mfg. Co., Newman, Ga.

Progress Made on Regional Plan for New York District

The remarkable progress that has been made toward reaching the ideal laid down in the regional plan for the New York Metropolitan District was reported to the Regional Plan Association by George McAneny, president of the association, at its fourth annual dinner held in New York on June 8.

In the four years since the plan was published, 555 miles of major highways, one-fifth of the mileage shown on the plan, have been either built, rebuilt or extensively widened or placed under construction, the mileage of the express highways has increased from 3.5 to 81 miles and there are 136 miles of new parkways and boulevards completed or under construction and 131 miles planned, a total equal to 28 per cent of the amount of such facilities called for in the plan. The area now has 22,000 acres of new public parks, of which 2,440 are within New York City, or one-twelfth of the park acreage recommended in the regional plan. Official planning boards have increased from 61 four years ago to 109 and eight counties have planning boards.

SOCIETY CALENDAR

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS, Chicago, Edgwater Beach Hotel, June 26-30.

AMERICAN SOCIETY OF AGRICULTURAL ENGINEERS, Annual meeting, Purdue University, Lafayette, Ind., June 21-24.

AMERICAN SOCIETY OF CIVIL ENGINEERS, annual convention, Chicago, Palmer House, June 27-30.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS, Chicago, Palmer House, June 27-July 1.

AMERICAN SOCIETY OF MUNICIPAL ENGINEERS, Milwaukee, Wis., Sept. 25-27.

AMERICAN SOCIETY FOR TESTING MATERIALS, annual meeting, Chicago, Stevens Hotel, June 26-30.

NATIONAL COUNCIL OF STATE BOARDS OF ENGINEERING EXAMINERS, annual convention, Chicago, Stevens Hotel, June 26-28, 1933.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION, annual meeting, Chicago, Stevens Hotel, June 27-30.

ENGINEERS WEEK, Century of Progress Exposition, Chicago, June 26-July 1.

AMERICAN SHORE & BEACH PRESERVATION ASSOCIATION will hold its mid-year meeting at the Hotel Commodore Perry, Toledo, Ohio, on June 23.

Personal Notes

EARLE B. PATTEN has been appointed assistant city engineer of St. Louis, Mo., to succeed Fred Wolffe.

J. A. ELLIOTT, assistant to the district engineer, U. S. Bureau of Public Roads, Portland, Ore., has been transferred to Denver, Colo., as acting district engineer to fill the vacancy created by the death of Junius W. Johnson.

WALTER M. SMITH, assistant chief engineer of the division of waterways of the Department of Purchase and Construction, State of Illinois, on June 2, was awarded the honorary degree of doctor of science by the Citadel, the military college of South Carolina.

W. F. KURFESS, manager of the mill department of Joseph T. Ryerson & Sons, has been appointed assistant vice president of the company. Mr. Kurfess is a graduate of Purdue University and entered the structural engineering department of the Ryerson Company in 1912.

JOHN H. PORTER, a member of the Missouri Public Service Commission, has tendered his resignation to take effect Aug. 1 in order that he may resume the private practice of engineering in St. Louis. He has been on the commission for five years. At present he is the only engineer member.

J. L. BURKHOLDER, engineer in charge of the forces of the Metropolitan Water District of Southern California constructing the Coachella tunnel division of the Colorado River aqueduct, has been appointed assistant general manager of the water district. The appointment was made by the district board, which created the position and named Mr. Burkholder on the recommendation of F. E. Weymouth, general manager and chief engineer.

GEORGE PARKMAN, JR., and WILLIAM H. KERR have organized the P-K Construction Co. with offices in the Fulton Building, Pittsburgh, Pa., to specialize in

reinforced concrete and brickwork and small building construction. For the past five years Mr. Parkman, a graduate civil engineer, has been with the Mellon-Stuart Co. of Pittsburgh. For the past eleven years Mr. Kerr has been with the same company and with the W. T. Grange Co. as engineer and superintendent.

Obituary

CHARLES E. VAIL, assistant chief engineer, department of water supply, gas and electricity, New York, died there on June 9, age 51 years. Mr. Vail had been in the water department of the city for 23 years.

JOSEPH B. MARCH, city engineer of Butler, Mo., and engineer for the Bates County drainage district, died at his home in Butler, Mo. on June 1, age 68 years. Mr. March was on the Panama Canal construction work for two years.

Brief News

DAMAGE CLAIMS amounting to $471,185 of the $477,901 claimed by the Whiting-Turner Construction Co. for extra work on the construction of the Montebello-Druid Lake water tunnel for the city of Baltimore have been disallowed by the Public Improvements Commission.

ACCEPTANCE OF A LOAN of $1,780,000 from the Reconstruction Finance Corp. by the Board of Regents of the Louisiana State University will not be possible, according to the legal advisers of the University, until the state supreme court passes upon the right of the board to pledge revenues of the university to the support of bonds or certificates issued to the R.F.C. in return for the loan. The loan was approved by the R.F.C. for use in the construction of buildings and expansion of university facilities.

A FEDERATION OF FLOOD CONTROL DISTRICTS between Pittsburgh, Pa. and Cincinnati, Ohio, was discussed at Gallipolis, Ohio, on June 7 by representatives of fourteen cities in Pennsylvania, Ohio, West Virginia and Kentucky. These representatives aim for the adoption of a national program for the conservation of the Ohio Valley similar to that set up by the federal government for the Tennessee Valley.

MOTION FOR A NEW TRIAL in the Southfield sewer case of the City of Detroit against Julius Porath and Son, contractors, and their bonding companies has been denied by the trial judge, Lester S. Moll, in circuit court. As a result of failures of sections of the Southfield sewer in 1930, the city brought action against the contractors for $355,000 damages on a $690,000 contract. In the motion for a new trial the city claimed that the verdict returned by a jury last March, awarding the city six cents damages was a compromise verdict. An appeal to the Michigan Supreme Court has been filed for the city.

APPORTIONMENT OF 40 PER CENT of the $693,000 cost of the Lindell-Union viaduct in St. Louis against the Wabash Railway is being protested by the receiver and counsel has asked the Missouri Public Service Commission for a rehearing. The railroad claims that it should not pay more than 10 per cent of the entire cost of the improvement, that the 40 per cent apportionment is an unlawful burden on interstate commerce.

NO BOND FOR THE COMPLETION OF CONTRACT was required of the successful bidder on the proposed bridge over the Hudson River at Catskill, N. Y., when the contract for its construction was let last week by the New York State Bridge Authority. Instead, the state will retain 20 per cent of the monthly estimate paid to the contractor. This change in the state practice was made upon the recommendation of Frederick Stuart Greene, superintendent of public works. A similar system was used in the construction of other bridges built recently by the state.

Contracts and Capital

CONSTRUCTION
(Thousands of Dollars)

	Weekly June, 1932	Average Four Weeks	Week June 15, 1933
Federal government	$6,384	$556	$166
State and municipal	12,002	5,955	5,345
Total public	$18,386	$6,511	$5,511
Total private	5,331	8,431	6,985
Week's total	$23,717	$14,942	$12,496
Cumulative to date:			
1932		$550,679	
1933		393,397	

NEW PRODUCTIVE CAPITAL
(Thousands of Dollars)

	Weekly June, 1932	Average Four Weeks	Week June 10 1933
New capital issues	$12,600	$5,752	$4,827
Cumulative to date:			
1932		$382,200	
1933		136,387	
R.F.C. self-liquidating loans:			
Approved current week			
Approved to date (including 1932)			$204,882
Actual loans made (including 1932)			47,195

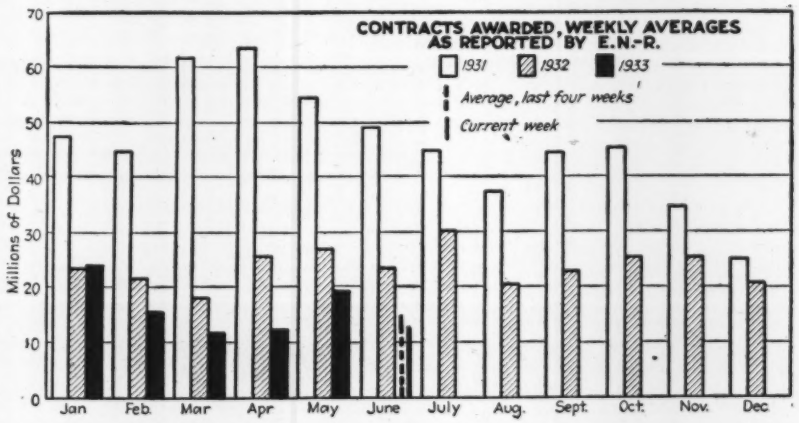
ENR Cost and Volume Index

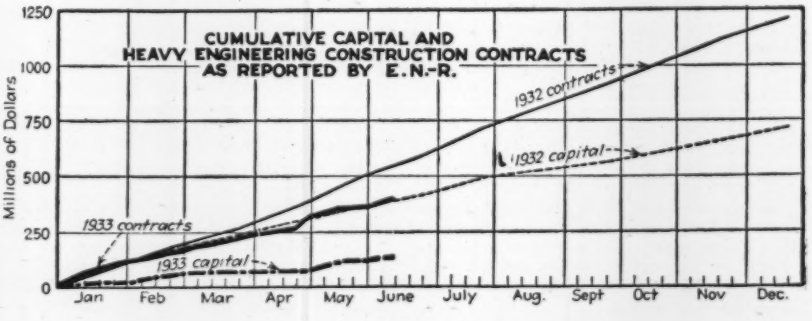
E.N.-R. Cost		E.N.-R. Volume	
June, 1933	**163.41**	**May, 1933**	**182**
May, 1933	164.39	April, 1933	65
June, 1932	152.20	May, 1932	149
1932 (Average)	156.97	1932 (Average)	127
1931 (Average)	181.35	1931 (Average)	220
1930 (Average)	202.35	1930 (Average)	260
1913 Average	100		

Engineering Contracts and Capital

HEAVY engineering construction contracts reported for the past week total $12,496,000, as compared with $21,511,000 the week previous. Private contracts held up well, amounting to $6,985,000, a loss of $600,000 from the previous week, but still ahead of the average for this class of work for the past two months. As was to be expected pending action of the public works bill, federal contracts were insignificant, totaling only $166,000 for the week. State and municipal awards were less than half those of the previous week, amounting to $5,511,000. Highway lettings have increased steadily for the past two months, reaching $3,800,000 last week. Highway awards in Missouri aggregated $1,453,000.

New productive capital issues sold last week totaled $4,827,000, all public issues. The R.F.C. announced approval of loans amounting to $2,811,000. The details of the loans are given on p. 791.





Unit Prices—Eleven Levee Sections, Mississippi

A TOTAL OF 58 bids were submitted in April by 27 bidders for the construction of eleven sections of Mississippi River relief levees in the Upper Yazoo district, Mississippi. A total of 5,843,000 cu.yd. of excavation is involved in the eleven sections. The work will be handled out of the Memphis District Engineer Office. Contracts as yet have not been awarded. Sections 271, 272, and 273 are near Hollywood, Miss. Sections 286 and 287 are about 4 miles west of Tunica, Miss. The remaining sections are northeast of Friars Point, Miss., in the Moon Lake district.

In the accompanying table are complete unit prices. Many contractors submitted individual bids for any or all sections, and in addition submitted lower bids for certain combinations of sections. These combinations are shown in the accompanying table, each combination being listed in a separate horizontal line. Low bids, shown in bold face type in the table, were submitted by the following firms: Sections 271, 272 and 273, combination bid for all three sections, 2,293,000 cu.yd., 11 c. per cu.yd., Brooks-Callaway Co., Atlanta; Sections 286 and 287, combination bid for both sections, 1,155,000 cu.yd., 10.5 c. per yd., Brooks-Callaway Co., Atlanta; Section 315A, 400,000 cu.yd., 13.5 c. per yd., Forcum James Co., Dyersburg, Tenn.; Sections 315B and 315C, 805,000 cu.yd., 12.7 c. per yd., Matthews Bros. Const. Co., Erie, Ill.; Section 315D, 405,000 cu.yd., 10.5 c. per yd., Harris & Brandenburg, West Memphis, Ark.; Sections 320 and 331 A, combination bid for both sections, 785,000 cu.yd., 11.98 c. per yd., Chas. W. Green, Opelousas, La. Many bidders stipulated total yardage they could handle. The table shows how the increased yardage of combinations of sections lowered the bid prices. For the first three sections, lowest bids exceeded the engineer's estimate. Bid table is continued on following page.

COMPLETE UNIT PRICES — ELEVEN LEVEE SECTIONS, MISSISSIPPI RIVER, UPPER YAZOO DISTRICT
(Low bids are indicated by **bold face** type)

Bidder	Number of Sections	Sec. L-271 766,000 cu.yd.	Sec. L-272 735,000 cu.yd.	Sec. L-273 792,000 cu.yd.	Sec. L-286 560,000 cu.yd.	Sec. L-287 595,000 cu.yd.	Sec. L-315A 400,000 cu.yd.	Sec. L-315B 400,000 cu.yd.	Sec. L-315C 405,000 cu.yd.	Sec. L-315D 405,000 cu.yd.	Sec. L-320 485,000 cu.yd.	Sec. L-331A 300,000 cu.yd.
Matthews Bros. Const. Co., Erie, Ill.	Individual	.....	.....	.....	$0.1675	$0.175	.....	$0.127	$0.127	.....	$0.16	$0.165
Matthews Bros. Const. Co., Erie, Ill.	Combination	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
M. W. O'Meara, Quincy, Ill.	Individual	$0.1071	$0.1071	$0.1071	.1091	.1091	$0.135	.135	.135	$0.135	.14	.13
M. W. O'Meara, Quincy, Ill.	Combination	.1065	.....	.....	.1065	.....	.....	.....	.....	.....	.....	.....
M. W. O'Meara, Quincy, Ill.	Combination	.....	.1065	.....	.1065	.....	.....	.....	.....	.....	.....	.....
M. W. O'Meara, Quincy, Ill.	Combination	.1065	.....	.....	.....	.1065	.....	.....	.....	.....	.....	.....
M. W. O'Meara, Quincy, Ill.	Combination	.....	.1065	.....	.....	.1065	.....	.....	.....	.....	.....	.....
M. W. O'Meara, Quincy, Ill.	Combination	.....	.....	.....	.1061	.1061	.....	.....	.....	.....	.....	.....
Winston Bros. Co., St. Louis	Individual	.....	.....	.....	.1675	.1675	.....	.....	.....	.....	.....	.1965
A. Guthrie & Co., St. Paul	Individual	.....	.....	.....	.....	.....	.....	.....	.....	.....	.15	.12
A. Guthrie & Co., St. Paul	Combination	.....	.....	.....	.....	.....	.....	.....	.....	.....	.1347	.1347
Geo. W. Condon Co., Omaha	Individual	.....	.....	.....	.139	.149	.164	.164	.164	.164	.169	.149
Geo. W. Condon Co., Omaha	Combination	.....	.....	.....	.139	.139	.154	.154	.154	.154	.....	.....
Geo. W. Condon Co., Omaha	Combination	.....	.....	.....	.139	.139	.....	.....	.....	.....	.....	.139
Ball & Jones, Fayette, Miss.	Individual	.1085	.1082	.1085	.108	.108	.11	.115	.115	.11	.124	.1487
Ball & Jones, Fayette, Miss.	Combination	.1083	.1083	.1083	.....	.....	.....	.....	.....	.....	.....	.....
Ball & Jones, Fayette, Miss.	Combination	.....	.....	.....	.1079	.1079	.....	.1122	.1122	.1122	.....	.....
C. E. Walker, Cedar Rapids, Iowa	Individual	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.175
Carl Erickson, W. Monroe, La.	Individual	.14	.14	.14	.15	.15	.165	.165	.145	.145	.15	.14
The Drivor Co., Memphis	Combination	.169	.169	.169	.....	.....	.....	.....	.....	.....	.....	.....
The Drivor Co., Memphis	Combination	.....	.....	.....	.1499	.1499	.....	.....	.....	.....	.....	.....
The Drivor Co., Memphis	Combination	.....	.....	.....	.....	.....	.158	.158	.158	.158	.....	.....
The Drivor Co., Memphis	Individual	.....	.....	.....	.....	.....	.....	.....	.....	.....	.163	.133

Asphalt Airport Runways, New Orleans

CONTRACT WAS LET in April, 1933, for the construction of 2.6 miles (152,000 sq.yd.) of asphalt surface runways for Shushan Airport, New Orleans. Bids were received on two alternate types of surfacing, Item B1 calling for sheet asphalt 4 in. compacted thickness, laid in two courses, a 3-in. base course and a 1-in. top course. Item B2 was specified as an alternate surfacing, consisting of 4 in. of compacted "Lay-Kold" sand-asphalt surfacing as manufactured by the American Bitumuls Co. Contract was awarded on the basis of the B2 alternate. Specifications called for plant-mix surfacing material, as the nature of the ground would not permit mix-in-place methods. A bonus and penalty clause was inserted in the specifications, calling for a penalty of $100 per day for every day beyond time of completion specified by the contractor in his bid, and likewise providing for a bonus of $100 per day for each day saved under the time specified. In tabulating the bids, a price of $100 per day was added to each bid over the minimum number of days specified by any contractor. Following are complete unit prices. (A) Wesco Co., Chattanooga, contract; (B) Southern Roads Co., Memphis; (C) Nelson Bros., Baton Rouge. John Klorer is chief engineer of the Board of Levee Commissioners, in charge of constructing Shushan Airport.

ASPHALT RUNWAYS, SHUSHAN AIRPORT

Item	Quan.	Bidders A	B	C
A — Fine grading, sq.yd.	152,000	$0.01	$0.02	$0.02
B1 — Hot sheet asphalt, sq.yd.	152,000	.65	.695	.....
B2 — Lay-Kold Surfacing, sq.yd.	152,000	.72	.735	.78
C — Seal coat and chip cover, sq.yd.	152,000	.10	.08	.15
Total, Items A, B1		$100,320	$108,680	.....
Total, Items A, B2		$110,960*	$114,760	$115,216
Days for completion		50	75	90
Corrected for time:				
Total, Items A, B1		$100,320	$111,180	.....
Total, Items A, B2		$110,960	$117,260	$119,216

*Contract.

Complete Unit Prices—Eleven Levee Sections, Mississippi (Continued)

Bidder	Number of Sections	Sec. L-271 766,000 cu.yd.	Sec. L-272 735,000 cu.yd.	Sec. L-273 792,000 cu.yd.	Sec. L-286 560,000 cu.yd.	Sec. L-287 595,000 cu.yd.	Sec. L-315A 400,000 cu.yd.	Sec. L-315B 400,000 cu.yd.	Sec. L-315C 405,000 cu.yd.	Sec. L-315D 405,000 cu.yd.	Sec. L-320 485,000 cu.yd.	Sec. L-331A 300,000 cu.yd.
Missouri Engineering & Constr. Co., St. Louis	Individual	.....	.....	.....	.....	.....	.....	.....	.....	.....	.184	.....
Yale-Reagan & Benson-Kaiser, St. Louis	Individual	.1495	.1495	.1495	.1495	.1495	.155	.165	.165	.165	.1695	.1495
Walter H. Denison, Cushman, Ark.	Individual	.1425	.1425	.1425	.1369	.1369	.155	.1523	.1523	.1523	.1523	.1399
Walter H. Denison, Cushman, Ark.	Combination	.....	.....	.....	.1349	.1349	.....	.....	.....	.....	.....	.....
J. H. Morrow, Greenville, Miss.	Individual	.142	.142	.143	.142	.142	.1455	.1455	.1455	.1455	.1469	.144
J. H. Morrow, Greenville, Miss.	Combination	.140	.140	.....	.....	.....	.....	.....	.....	.....	.....	.....
J. H. Morrow, Greenville, Miss.	Combination	.....	.....	.....	.1389	.1389	.....	.....	.....	.....	.....	.....
J. H. Morrow, Greenville, Miss.	Combination	.....	.....	.....	.....	.....	.....	.1419	.1419	.....	.....	.....
J. H. Morrow, Greenville, Miss.	Combination	.....	.....	.....	.....	.....	.....	.....	.....	.....	.1445	.1445
J. H. Morrow, Greenville, Miss.	Combination	.1382	.1382	.1382	.....	.....	.....	.....	.....	.....	.....	.....
J. H. Morrow, Greenville, Miss.	Combination	.....	.....	.....	.....	.....	.1393	.1393	.1393	.1393	.....	.....
J. H. Morrow, Greenville, Miss.	Combination	.....	.....	.....	.....	.....	.1419	.....	.....	14.19	.....	.....
J. H. Morrow, Greenville, Miss.	Combination	.....	.....	.....	.1414	.1414	.....	.....	.....	.....	.1414	.1414
J. H. Morrow, Greenville, Miss.	Combination	.....	.....	.....	.....	.....	.1456	.....	.....	.1456	.1456	.1456
Council & Lewy Co., Greenville	Individual	.....	.....	.....	.....	.....	.16	.16	.16	.16	.16	.16
Harris & Brandenburg, West Memphis	Individual	.105	.105	.105	.105	.105	.12	.11	.11	.105	.125	.13
Harris & Brandenburg, West Memphis	Combination	.....	.....	.....	.....	.....	.....	.....	.....	.....	.125	.125
Callahan-Walker Const. Co., Omaha	Combination	.154	.154	.154	.154	.154	.154	.154	.154	.154	.....	.....
Callahan-Walker Const. Co., Omaha	Combination	.154	.154	.154	.154	.154	1.54	.154	.154	.154	.154	.....
Callahan-Walker Const. Co., Omaha	Combination	.154	.154	.154	.154	.154	.154	.154	.154	.154	.154	.154
C. J. List Const. Co., Kansas City	Individual	.137	.134	.134	.138	.139	.157	.152	.15	.15	.16	.14
C. J. List Const. Co., Kansas City	Combination	.....	.....	.....	.....	.....	.153	.153	.....	.....	.....	.....
C. J. List Const. Co., Kansas City	Combination	.....	.....	.....	.....	.....	.....	.....	.1495	.1495	.....	.....
C. J. List Const. Co., Kansas City	Combination	.....	.....	.....	.....	.....	.....	.15	.15	.....	.....	.....
Forcum James Co., Dyersburg, Tenn.	Individual	.1375	.1375	.135	.131	.134	.135	.145	.145	.135	.15	.13
Forcum James Co., Dyersburg, Tenn.	Combination	.....	.....	.....	.131	.131	.....	.....	.....	.....	.....	.....
Forcum James Co., Dyersburg, Tenn.	Combination	.....	.....	.....	.....	.....	.....	.1425	.1425	.....	.....	.....
Forcum James Co., Dyersburg, Tenn.	Combination	.....	.....	.....	.....	.....	.....	.....	.....	.....	.135	.136
Brooks-Callaway Co., Atlanta	Individual	.....	.....	.....	.....	.....	.....	.....	.....	.....	.15	.15
Brooks-Callaway Co., Atlanta	Combination	.11	.11	.11	.....	.....	.....	.....	.....	.....	.....	.....
Brooks-Callaway Co., Atlanta	Combination	.....	.....	.....	.105	.105	.....	.....	.....	.....	.....	.....
Brooks-Callaway Co., Atlanta	Combination	.....	.....	.....	.....	.....	.15	.15	.15	.15	.....	.....
Trinity Farms Const. Co., Dallas	Individual	.1562	.1569	.1564	.1617	.1896	.....	.....	.....	.....	.....	.1946
Chas. W. Green, Opelousas, La.	Individual	.1144	.1144	.1144	.1198	.1198	.1434	.1434	.1434	.1434	.1198	.15
Chas. W. Green, Opelousas, La.	Combination	.....	.....	.....	.1194	.1194	.....	.....	.....	.....	.....	.....
Chas. W. Green, Opelousas, La.	Combination	.....	.....	.....	.....	.....	.1204	.1204	.....	.....	.....	.....
Chas. W. Green, Opelousas, La.	Combination	.....	.....	.....	.....	.....	.....	.....	.1198	.1198	.....	.....
Chas. W. Green, Opelousas, La.	Combination	.....	.....	.....	.....	.....	.....	.....	.....	.....	.1198	.1198
Chas. W. Green, Opelousas, La.	Combination	.....	.....	.....	.....	.....	.12	.12	.....	.....	.....	.....
Henry Hughey Co., Inc., Memphis	Individual	.....	.....	.....	.143	.144	.184	.18	.18	.17	.169	.142
Horton Price Const. Co., Lake Providence, La.	Combination	.153	.153	.153	.....	.....	.....	.....	.....	.....	.....	.....
Horton Price Const. Co., Lake Providence, La.	Combination	.....	.147	.147	.....	.....	.....	.....	.....	.....	.....	.....
C. C. Aderholt, Friars Point, Miss.	Individual	.15	.15	.15	.15	.15	.1626	.13	.13	.1326	.1539	.1626
Sternberg Co., Inc., St. Louis	Individual	.....	.....	.....	.....	.....	.....	.....	.....	.....	.2275	.22
Sternberg Co., Inc., St. Louis	Combination	.....	.....	.....	.1885	.1885	.....	.....	.....	.....	.....	.....
Gaines Bros. Co., Fairland, Okla.	Individual	.16	.16	.17	.17	.17	.18	.18	.18	.18	.16	.18
M. Wunderlich, Jefferson City, Mo.	Individual	.14	.14	.14	.14	.14	.145	.145	.145	.145	.147	.145
M. Wunderlich, Jefferson City, Mo.	Combination	.138	.138	.....	.....	.....	.....	.....	.....	.....	.....	.....
M. Wunderlich, Jefferson City, Mo.	Combination	.....	.....	.....	.138	.138	.....	.....	.....	.....	.....	.....
M. Wunderlich, Jefferson City, Mo.	Combination	.....	.....	.....	.....	.....	.....	.14	.14	.....	.....	.....
M. Wunderlich, Jefferson City, Mo.	Combination	.....	.....	.....	.....	.....	.....	.....	.....	.....	.144	.144
M. Wunderlich, Jefferson City, Mo.	Combination	.138	.138	.138	.....	.....	.....	.....	.....	.....	.....	.....
M. Wunderlich, Jefferson City, Mo.	Combination	.....	.....	.....	.....	.....	.14	.....	.....	.14	.....	.....
Government Estimate	Individual	.1139	.1134	.1206	.1354	.1339	.1666	.1571	.1599	.1624	.161	.1534
Government Estimate	Combination	.1069	.1069	.1069	.....	.....	.....	.....	.....	.....	.....	.....
Government Estimate	Combination	.....	.....	.....	.1334	.1334	.....	.....	.....	.....	.....	.....
Government Estimate	Combination	.....	.....	.....	.....	.....	.....	.1424	.1424	.....	.....	.....
Government Estimate	Combination	.1089	.1089	.....	.....	.....	.....	.....	.....	.....	.....	.....

Bids marked Individual are for any or all sections. Bids marked Combination are for total of all sections listed in each horizontal line.